APPLICATION OF SOIL LIQUEFACTION PREDICTIVE MODELS TO SOME SITES IN UTTAR PRADESH AND THEIR COMPARATIVE STUDY

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Dedicated to
my Father

CERTIFICATE

This is to certify that the thesis entitled, "*Application of Soil Liquefaction Predictive Models to Some Sites in Uttar Pradesh and Their Comparative Study*", by Manoj Kumar Rajpal is a record of work carried out by him under my supervision and has not been submitted elsewhere for a degree.

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ABSTRACT

The effect of soil liquefaction during earthquake is disastrous and has great social and economic impact on the affected population. As such, a systematic study of liquefaction potential of different earthquake prone areas of country is necessary for adopting preventive measures. Large tract of alluvial deposits of the Ganga basin lies in the tectonically active region and is likely to liquefy.

So, there is scope for mapping of liquefaction potential of various sites in these deposits. With this in mind, an attempt has been made in this thesis to develop a computer program where in the liquefaction evaluation of a site can be evaluated by using different approaches proposed by various investigators. The program has the capability to choose appropriate method or methods for analysing the data collected for the purpose. The program so developed has been used to find out the liquefaction potential of different sites of Uttar Pradesh.

Based on the study conducted, the sites can be sequentially placed in the decreasing order of risk of soil evaluation as Anola, Shahjahnpur, Deoria, Gorakhpur, Raibarely and Kanpur. Comparative study of the various method revealed that Chang predictive model for soil liquefaction evaluation is in general the most conservative. Shibata and Teparakasa and Bolton and Idriss also predict values on the conservative side. Other models predict large factor of safety against soil liquefaction.

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Manoj Kumar Rajpal

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LIST OF SYMBOLS

α_{max}	:	Maximum ground acceleration
a	:	Empirical constant
C	:	Clay content
C_r	:	Correction factor to be applied to laboratory triaxial test data to obtain the stress condition causing liquefaction in the field
C	:	Empirical parameter depending on strain amplitude
DA	:	Double amplitude of axial strain
D_r	:	Relative density
D_{50}	:	Mean grain size
ds	:	Depth to sand or sandy loam layer under consideration
dw	:	Depth of water table below ground level
F_l	:	Liquefaction resistence factor
FC	:	Fine content
g	:	Acceleration due to gravity
h,z	:	Depth below groun level
IP	:	Plasticity index
L	:	Cyclic load applied
M	:	Earthquake magnitude
n,Δnf	:	Empirical constant
N	:	Uncorrected SPT value
N_1	:	Corrected SPT value
PC	:	Clay content
q_c	:	Uncorrected CPT value
q_{c1}	:	Corrected CPT value

$(q_{c1})_{cr}$	:	Uncorrected critical CPT value
$(q_c)_{cr}$	:	Corrected critical CPT value
qc	:	Static cone penetration value
$\overline{q}c$	:	Critical static cone penetration valueρ
γ	:	Unit weight of soil
R_1	:	Cyclic undrained triaxial strength
R	:	Resistence of soil element
γ_n	:	Correction factor
$(\tau_{max})_{\gamma}$	:	Maximum shear stress on the soil element
$(\tau_{max})_d$		Actual shear stress
τ_{av}	:	Average cyclic shear stress
$(\tau/\sigma_o^{'})_1$	:	stress ratio causing liquefaction
$(\tau_d/\sigma_o^{'})_{20}$	:	Cyclic stress ratio required to cause liquefaction in 20 cycles
τ_d	:	Amplitude of uniform shear stress cycles equivalent to actual seismic shear stress time history
$(\sigma_{dc}/2\ \sigma a)_1$	:	Stress ratio causing liquefaction in laboratory cyclic triaxial tests
σ_{dc}	:	Cyclic deviator stress
σ_a	:	Initial ambient pressure under which sample was consolidated
$\sigma_o^{'}$	:	Effective stress
σ_o	:	Total stress
Vs	:	Wave velocity value
$\overline{V}s$	:	Critical wave velocity

CHAPTER 1

INTRODUCTION

1.1 General:- It is widely recognized that the basic mechanism of liquefaction in a deposit of loose saturated sand during earthquake is the progressive built up of excess pore water pressure due to the application of cyclic shear stresses induced by the upward propagation of shear waves from the underlying rock formation. Under ordinary conditions prior to an earthquake, a soil element in the level ground is subjected to a confining stress due to the weight of the overlying soils. When a series of cyclic stress is applied during an earthquake, the elements of loose sand tends to reduce its volume. However, since the duration of cyclic stress application is so short as compared to the time required for drainage of water, the volume contraction can not occur immediately. In order to keep the contracting loose sand at a constant volume some change in the existing stress system must take place. This stress change is achieved in the form of a reduction in the existing confining stress and a consequent increase of equal magnitude of the pore water pressure. Therefore the degree of the pore water pressure increase depends, on one hand upon the state of packing indicative of potential of the volume decrease tendency and on the other hand, upon how far the sand is sheared to extract the inherent volume decrease characteristics. Fig-1.1(a) indicates the deposit prior to liquefaction.

When the state of sand packing is loose enough and the

magnitude of cyclic shear stress is great enough, the pore water pressure builds up to a full extent in which it becomes equal to the initially existing confining stress. At this state, no effective stress or intergranular stress is acting on the sand and individual particles released from any confinement exist as if they were floating in water as illustrated in fig- 1.1(b). Such a state is called liquefaction.

Upon occurrence of liquefaction, individual particles of the sand start to sediment in water thereby expelling pore water towards the surface of the sand deposit;when the sedimentation has taken place throughout the depth of the deposit, the sand is deposited in a somewhat denser state, as shown in fig-1.1(c). The transfer of the state of sand from the initial deposition to the final dense state via the process of liquefaction is illustrated in fig- 1.1, in which scale inside the box is assumed to indicate the effective stress and the outside scale supporting the sand filled box indicates the total stress.

The length of the time for which the liquefaction state continues to exist depends upon drainage condition of the deposit and also on the duration of cyclic shear stress application following the onset of liquefaction. The longer and the stronger the cyclic shear stress application, the longer the state of liquefaction persists; the thicker the deposit and the finer the sand composing the deposit, the longer the time required to drain the developed excess pore water pressure and therefore the longer is the state of liquefaction

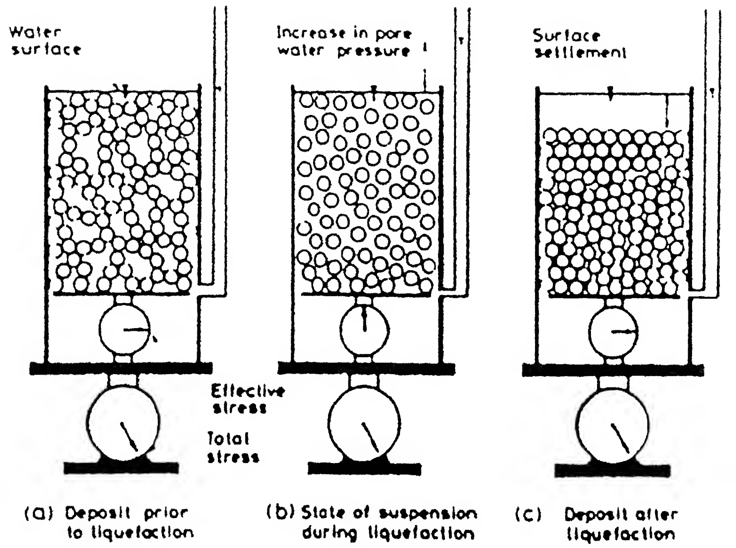

Fig. 1.1 Transfer of state of deposition via liquefaction

Thus in short we can say that the act or the process of transforming soil into a liquid state is called liquefaction.

1.2 Effects of liquefaction

The major earthquakes that occurred in Nigata ,Japan and in Alaska in 1964 showed that great damage can result from high seismically induced pore water pressure and soil liquefaction. The damage caused by earthquake can be enormous. The Alaska earthquake had enormous social and economic impact to the residents of Alaska including 114 causalities. Public and private property losses were estimated as $311 million (1964 value). Lifelines including the transportation system were disrupted for months following the events.

The intense shaking triggered numerous destructive ground failures including flow failures, lateral spreads and landslides. Lateral spreads were particularly destructive to highway and railways bridge within a 130km radius of the zone of energy release. Total damage to the highway system was $46 million (1964 value); $25 million was incurred by highway bridge and $21 million by roadways. To highlight the disastrous effect due to soil liquefaction during earthquake several examples such as Borego mountain earthquake (1968), Tokachioki(1968), Sen ferando(1971), Haicheng and Tangshan earthquake(1975), Vrancea(1977), can also be cited.

1.3 Literature review

Terzaghi(1925) was the first, who dealt with the phenomenon of liquefaction when he watched such an occasion in the province

of Zealand in the Netherlands. Later Casagrande(1936) conducted experiments to study the effect of soil density on the cyclic strength of the soil.

Until the late 1960's when an engineer had to estimate whether the soils at a particular site might liquefy during an earthquake or not, the major tool available to him was the standard penetration test in conjunction with several empirical correlations. For example, Ohsaki(1970) describes the useful Japanese rule of thumb that liquefaction is not a problem if the blow count from a standard penetration test exceeds twice the depth of sample measured in meters. He also provides a unique case history in which the liquefaction was predicted and designed, and in which the prediction were verified during a subsequent earthquake.

After the enormous damage due to soil liquefaction during the Alaska and Nigata earthquakes extensive studies were made with emphasis on the observations of field behaviour and laboratory modelling. This led to the development of liquefaction predicting models which correlated the actual field behaviour to some soil parameters. As the disastrous effect of the liquefaction had been realized by the geotechnical engineering society, efforts were made to predict the liquefaction potential of a site so that appropriate measures could be taken if there was any risk of liquefaction.

A simplified procedure proposed by seed et al.(1971) by which a series of uniform cyclic stresses, assumed to be

equivalent in their effect to the irregular stress sequence produced by an earthquake could be determined

The number of stress cycles over which the equivalent uniform shear stress is repeated may be evaluated either by using an appropriate weighting procedure or by adopting a representative number of cycles from studies of different magnitude as presented by Lee et al.(1972).

Typical number of cycles first presented by seed et al.(1971) was updated by Lee et al.(1972). In all the cases, the number of cycles corresponding to an equivalent uniform shear stress (usually 0.65 τ_{max}) has been plotted as a function of the earthquake magnitude. Another simplified procedure that may be used to evaluate the liquefaction potential of a horizontal soil deposit is based on the concept that the effect of cyclic loading on a soil is analogous to fatigue effect in structural materials. This method has been described by Donovan(1971) and may be referred to as a cumulative damage method since it makes use of Minor's damage equation expressed in an integral form.

Shibata et al.(1972) performed a series of quick cyclic shear tests on saturated sands of low density in the vibratory triaxial apparatus keeping the mean principal stress constant. They pointed out that pore water pressure increment which accumulates during one loading cycle before the on-set of initial liquefaction may be expressed as a function of the octahedral shear stress and mean principal stress or that of shear stress and normal stress on the shear plane.

Ishihara and Li(1972) performed strain controlled cyclic torsion tests on saturated sand using a triaxial torsion apparatus in which the vertical piston was designed so that its cross sectional area was equal to that of the specimen. In the first test cyclic twist was applied keeping the principal stress ratio constant. In this liquefaction did not occur. In the second test a sample consolidated anisotropically and subjected to cyclic twist was found to develop pore pressure faster than sample consolidated isotropically.

Ishihara and Yasuda(1972) performed dynamic triaxial shear tests on Nigata sand employing the time histories of axial stress chosen to be similar to those of accelograms recorded at the time of Nigata earthquake. It was observed that liquefaction could occur 9 seconds after the initiation of main shaking.

Yoshmi and Kuwabara(1973) concluded that if subsurface soil is liquefied then surface soil can also be liquefied if its permeability and compressibility are sufficiently low as compared to those of initially liquefied soil.

Ishibashi and Sherif(1974) used torsion shear device to find liquefaction. They observed that when the test data are plotted in the form($\Delta\tau_{max}/\sigma_{oct}$) versus the number of cycles to liquefaction, the initial value of K_o does not influence the liquefaction potential of soil.

Ishihara et al.(1975) proposed a model by which it is possible to assess the development of pore pressure and shear

strains in an element of sand when it undergoes cyclic loading in undrained conditions. The performance of sand predicted by this method proved relevant and useful to the assessment of dynamic pore pressure development and consequent occurrence of liquefaction for wide range of loading conditions from static to dynamic stress controlled to strain controlled, and from uniform to irregular cycles of excitation.

Ishihara and Yasuda(1975) performed dynamic torsion shear tests on hollow cylindrical specimens of saturated sands using irregular time histories of loading. They found that shock type of loading pattern gives greater resistance to liquefaction than the vibration type of loading.

Yoshmi and Oka (1975) conducted a series of dynamic cyclic shear tests under undrained condition using a ring torsion apparatus to apply cyclic shear stresses under nearly plain strain conditions. They observed that irrespective of the magnitude of initial shear stress, liquefaction failure is imminent when ratio of the peak shear stress to the vertical effective stress reached a critical value of approximately 0.30 which coincided with the static failure condition. The coefficient of earth pressure at rest was evaluated using Jaky's relationship. As the initial shear stress was increased, smaller dynamic shear was required to cause initial liquefaction in a given number of stress cycles.

Ishihara et al.(1976) proposed a computational scheme for tracing the progressive increase of the pore pressure which

develop in sand deposits during earthquake.

Silver and Park(1976) developed a method for evaluating liquefaction potential of loose to medium dense sands from strain-controlled triaxial tests. It was found that liquefaction potentials determined from stress-controlled liquefaction tests were in good agreement for medium dense sand. For the same material at the same density, values of modulus and liquefaction potential are significantly higher for specimens prepared using rodding methods than for specimens prepared using dry vibration methods.

Ishihara and Watanabe(1976) performed cyclic triaxial shear tests on granular materials with a variety of grain size and uniformity coefficients to determine the effect of grain composition on the liquefaction potential of cohessionless soils. It has been shown that the effect of density on liquefaction potential on sand can be evaluated better in terms of difference between the current minimum void ratios than in terms of conventional measures of relative density. The new parameter, e-emin, called volume decrease potential implies that the farther a sand at its present state of packing can be compacted until it reaches its densest state, the more the sand is to liquefy.

Finn et al.(1976) presented a method for computing the dynamic response of a saturated sand stratum to earthquake motion in terms effective stress where in the dynamic properties of the sand had been modified for the effect of dynamic shear strains and progressive increase in the pore water pressure. The method

helped in computing the distribution of acceleration and shear stresses in a saturated sand during an earthquake.

Alba et al.(1976)performed large scale simple shear tests and pointed that initial liquefaction, a condition where the dynamically induced pore water pressure is equal to initial vertical effective stress may be induced even in dense deposits during large magnitude earthquake.

Ishihara et al.(1977) based on undrained cyclic shear experiments conducted on a saturated Ottawa sand in torsion simple shear device, predicted pore pressure rise in the soil under uniform and non uniform dynamic shear stresses. They studied the effect of the stress loading history, number of stress cycles and stress intensity function.

Ishihara(1977) established a relationship between cyclic stress ratio and the residual pore water pressure by conducting cyclic triaxial tests on saturated sample of sands. This relationship permits pore water pressure to be assessed that will develop in a horizontal sand deposit during a given shaking during of earthquake. He also evaluated the factor of safety against liquefaction in terms of the depth of deposit.

Mulilis et al.(1977) investigated the effects of the method of sample preparation on the liquefaction characteristics of remoulded samples of saturated sands under stress controlled cyclic test conditions. They found that liquefaction characteristics of samples of saturated sand, remoulded by

different compaction procedure to the same density, may be significantly different. Fabric studies and electrical conductivity measures indicate that differences in the orientation of the contacts between sand grains and in packing were probably the primary reasons for the observed differences in the dynamic strength of sand.

Seed et al.(1977) studied the effect of seismic history on the liquefaction characteristics of saturated sands. It was found both analytically and experimentally that deposit of sand subjected to low magnitude earthquake, which are not sufficiently strong to cause liquefaction, will develop an increased resistance to liquefaction in subsequent earthquake even though they may undergo no significant change in density. This increased resistance may be due to change in structure of the sand system or an increase in the lateral earth pressure coefficient K_o.

Considering the non linear behaviour Finn et al.(1977) developed a method to find the dynamic response of dry or saturated sands. The method consists of a procedure for dynamic analysis with a specific stress-strain law and a method for computing volume changes and pore water pressure concurrently with the dynamic response.

Liou et al.(1978) developed a numerical model for liquefaction in a level or nearly horizontal deposit and demonstrated the potential applicability of the model by case studies related to the Nigata earthquake of 1964.

Ishihara and Okasa(1978a) performed static undrained triaxial shear tests on sand specimens, over consolidated to OCR values of 1.0 to 5.0, under cyclic as well monotonous loading conditions. It was found that pore water pressure and shear strains which developed during undrained shear tests were lower for over consolidated sand than for normally consolidated sand even when the density of sand was kept virtually unchanged.

Tatsuoka et al.(1978) proposed a new method on the basis of data from sand sampling procedure and dynamic triaxial tests on undisturbed sample, for evaluation of dynamic shear strength of sands from N-values by SPT and D_{50} value. In this D_r was not used thus reducing uncertainties.

Ishihara and Okasa(1978b) investigated the possible effects of preshearing on the cyclic behaviour of saturated sands. it was found that samples subjected to small preshearing developed less pore water pressure and shear strains on both sides of triaxial compression and extension. It was discovered that the sample subjected to large preshear on one side of triaxial loading, compression or extension, became stiffer on that side, but softer on opposite side.

Youd and Perkins(1978) developed a procedure for using geologic and seismologic information in compiling maps showing liquefaction induced ground failure potential.

Martin et al.(1978) studied the effect of system compliance on liquefaction tests. Compliance arises if the volume of the

confining samples increases as the pore water pressure increases. A compliance leads to an increase in stress ratio causing initial liquefaction.

Ishihara and Takatsu(1979) suggested an empirical formula to evaluate the effect of the over consolidation ratio as well as increased K_o value on the liquefaction resistance of sand deposits by conducting a series of cyclic torsion shear tests on clean sands over consolidated to different degrees under various (K_o) conditions. It was found that cyclic stress ratio causing initial liquefaction in a given number of cycles increases approximately in proportion to the square root of the over consolidation ratio in the range of OCR value of 1.0 and 4.0 irrespective of the K_o value.

Tatsuka et al.(1980a) proposed a relationship between cyclic undrained strength for a uniform loading with a limited number of cyclic loading and that for a random cyclic loading.

Tatsuoka et al.(1980b) gave a simplified procedure for assessing soil liquefaction potential based on N-value. It was found that an empirical relation for evaluation of strength from N-values which takes into account of the effects of grain size on N-values could provide reasonable results. It was also found that when the effect of grain size on N-value were not taken into account, values of strength evaluated from N-values could be considerably smaller for fine silty sand.

Haldar(1980) gave a decision analysis framework to study

the liquefaction problem as an aid in selecting the most desirable solution for particular project considering both the technical and economic aspects of the problem.

Oka et al.(1981) developed a method of analysis of the liquefaction of saturated sand deposits using the stress-strain relation of sand . They found that permeability has a great influence on the response of the ground during an earthquake which causes liquefaction progressively. The distribution of excess pore water pressure depends on the permeability of soil deposits.

Nova and Haeckel(1981) proposed a constitutive model which may explain, in a phenomological sense, the generation of pore pressure in an undrained test either when the load is monotonically increased to the failure or when the sand is cyclically sheared at constant stress or strain amplitude.

Ghaboussi and Dickmen(1981) presented a method of analysis for evaluation of seismic response and liquefaction of horizontally layered ground subjected to multidirectional shaking.

Kagawa and Kraft(1981) proposed a model employing some simplifying assumptions which predicted with good accuracy the pore pressure response on both uniform and irregular stress loadings.

Fardis et al.(1982) gave a probabilistic methodology for

liquefaction analysis based on calculation of seismic stresses in the soil and laboratory experiments.

Vaid and Chern(1983) studied the influence of static shear on undrained cyclic loading behaviour of Ottawa sand and showed that presence of static shear does not always lead to increased resistance to liquefaction.

Tokimatsu and Yoshmi(1983) gave relationship between adjusted dynamic shear stress ratio and normalized SPT N-value.They also conducted laboratory tests on undisturbed sands . They observed that sand containing more than 10% fines has much greater resistance to liquefaction than clean sands having the same SPT N-values and extensive damage would not occur for clean sands with SPT N-value greater than 25 and silty sands containing more than 10% with SPT value greater than 20 .

Dickmen and Ghaboussi(1984) proposed an effective stress method of analysis for determination of seismic response and liquefaction of horizontally layered level ground. Saturated sand was modelled as saturated porous deformable media and the dynamic coupled equation so developed were solved. A new hysteretic model and an effective stress path model have been used.

Hara et al.(1985) presented the theoretical and experimental studies on the pore pressure developed in saturated sand subjected to cyclic shear tests under partially drained conditions and found that the rate of pore pressure build up under partially drained condition is different from the rate of

pore pressure build up under undrained conditions.

Robertson and Campanella(1985) presented correlation between cyclic stress ratio to cause liquefaction and modified cone bearing for sand and silty sand.

Seed et al.(1985) studied the influence of SPT procedures in soil liquefaction resistance. The field data has been reinterpreted and plotted in terms of a newly recommended standard $(N_1)_{60}$, determined in SPT tests. Energies associated with different methods of performing SPT tests in different countries and with different equipment have been summarized in tabular form for easy conversion of in any measured N value to standard $(N_1)_{60}$ value.

Tokimatsu and Nakamura(1987) considered the effect of membrane penetration on the generation of pore pressure in liquefaction tests. The major effect of membrane was to increase the number of cycles to cause liquefaction . The cyclic shear ratio was found to be unique function of membrane compliance ratio independent of applied shear stress.

Arulanandan and Muraleetharan(1988) utilized the electrical properties of soils to characterize their grain and aggregate properties used as basic electrical indices which are necessary to characterize the grain and aggregate properties of a soil. Average formation factor is a function of porosity. Anisotropy index is a function of soil fabric and in uncemented soil the average shape factor is a function of particle shape. Theoretical

equations relating these indices to the grain and aggregate properties of soil were presented. Theoretical relationship and correlations between mechanical properties, such as porosity, compression index, swelling index relevant to analysis of level ground soil liquefaction and combination of basic electrical properties are given.

Shibata and Teparaska(1988) used CPT value and grain size of the soils to find liquefaction potential. The field correlation between the earthquake induced cyclic stress ratio and normalized cone resistance indicates that fine grain size of $D_{50} \leq 0.25mm$ have greater resistance to liquefaction than do clean sands ($D_{50} \geq .25mm$) having the same q_c value.

Chinese code(1989) have also given the critical N value which separates liquefaction from non-liquefaction.

1.4 Motivation and scope of the work

Though the importance of liquefaction studies had been felt as early as 1925, it was only after 1964 that the topic became one of the thrust areas in the field of Geotechnical Engineering. Extensive experimental research has been carried out to understand the principles of liquefaction and the factors affecting it. Simultaneously attempts have been made to develop theoretical predictive models to forecast the possibilities of liquefaction in saturated sand deposits so that appropriate preventive measures could be taken in situations where liquefaction is likely to occur. Side by side based on field

had also been developed to predict the liquefaction potential of a site. With all these efforts the subject has developed to such an extent that one can predict about the liquefaction susceptibility of a saturated sand deposit during earthquake loading, with a fair degree of accuracy. As the effect of liquefaction during the earthquake is disastrous and has great social and economic impact on the affected population. It is very prudent to have a systematic study of the liquefaction potential of different earthquake prone areas of a country. Government agencies of different countries have in fact undertaken such projects.

A large tract of the alluvial deposits of the Ganga basin lies in the tectonically active region. The strata is such that the soil deposit is likely to liquefy during earthquake. During the recent Bihar earthquake such liquefaction had been observed to occur and reported (Jain et al.,1991). However, to the best of the author's knowledge apart from just reporting about its occurrence no systematic scientific and technological investigation had been carried out about the liquefaction potential of the various deposits in earthquake prone areas. As such, it has been felt that there is a scope for mapping of liquefaction potential of various seismic prone areas of the country. With this in mind an attempt has been made to develop a computer program where in the liquefaction potential of a site can be evaluated by using different approaches proposed by various investigators, based on the input data and the type of available test data. The program has the capability to choose appropriate method or methods for the analysis of the data.

Finally using the program so developed the liquefaction potential of some areas of the state of Uttar pradesh has been evaluated. A comparative study of the different methods has also been made.

The present chapter of the thesis deals with the introduction where in the principle of the liquefaction and its mechanism, a brief literature review, motivation and scope of the work has been discussed.

In chapter 2 the methods for which the computer program has been developed have been presented.

In chapter 3 the soil strata of various areas of Uttar pradesh for which liquefaction potential studies are made is presented. An empirical method for evaluating the ground acceleration during earthquake has also been discussed in this chapter.

In chapter 4 the liquefaction potential of the different site has been evaluated, generalised conclusions have been drawn and scope of the future work has also been presented.

CHAPTER 2

PREDICTIVE MODELS FOR DETERMINING LIQUEFACTION POTENTIAL

2.1 General:- In this chapter, various empirical predictive models suggested by various investigators to find the possibility of soil liquefaction during earthquake has been presented and discussed as follows.

2.2 Bolton and Idriss theory (1971):-

They suggested that the shear stresses developed at any point in a soil deposit during an earthquake appear to be primarily due to the upward propagation of shear waves in the deposit. If the soil column above a soil element at depth h behaved as a rigid body and the maximum ground surface acceleration were a_{max}; the maximum shear stress on the soil element would be

$$\left(\tau_{max} \right)_{\gamma} = \frac{\gamma \times h}{g} \times a_{max} \qquad (2.2.1)$$

In which γ = the unit weight of the soil

Because the soil column actually behaves as a deformable body, the actual shear stress at depth h, $(\tau_{max})_d$, as determined by a ground response analysis will be less than $(\tau_{max})_{\gamma}$ and might be expressed by

$$\left(\tau_{max} \right)_{d} = \gamma_d \times \left(\tau_{max} \right)_{\gamma} \qquad (2.2.2)$$

In which γ_d = a stress reduction coefficient with a value less than 1. Computation of the value of γ_d for a wide variety of earthquake motions and soil conditions having sand in the upper 50ft(15.24mt) have shown that γ_d falls with in the range of values shown in Fig-2.2.1.

It may be seen that in the upper 30 ft (9.14 mt) or 40 ft (12.2 mt) depth the scatter of the results is not great and for any of the deposits the error involved in using the average value shown by the dashed line would generally be less than about 5 %.

Thus to depths of about 40ft (12.2 mt), a reasonably accurate assessment of the maximum shear stress developed during an earthquake can be made from the relationship.

$$\left(\tau_{max} \right)_{\gamma} = \frac{\gamma \times h}{g} \times a_{max} \times \gamma_d \qquad (2.2.3)$$

It has been found with a reasonable degree of accuracy that the average equivalent uniform shear stress, τ_{av} ,is about 65% of the maximum shear stress τ_{max}. Combining this result with the preceeding expression for τ_{max}, it follows that for practical purposes the average cyclic shear stress may be determined by

$$\tau_{av} = 0.65 \times \frac{\gamma \times h}{g} \times a_{max} \times \gamma_d \qquad (2.2.4)$$

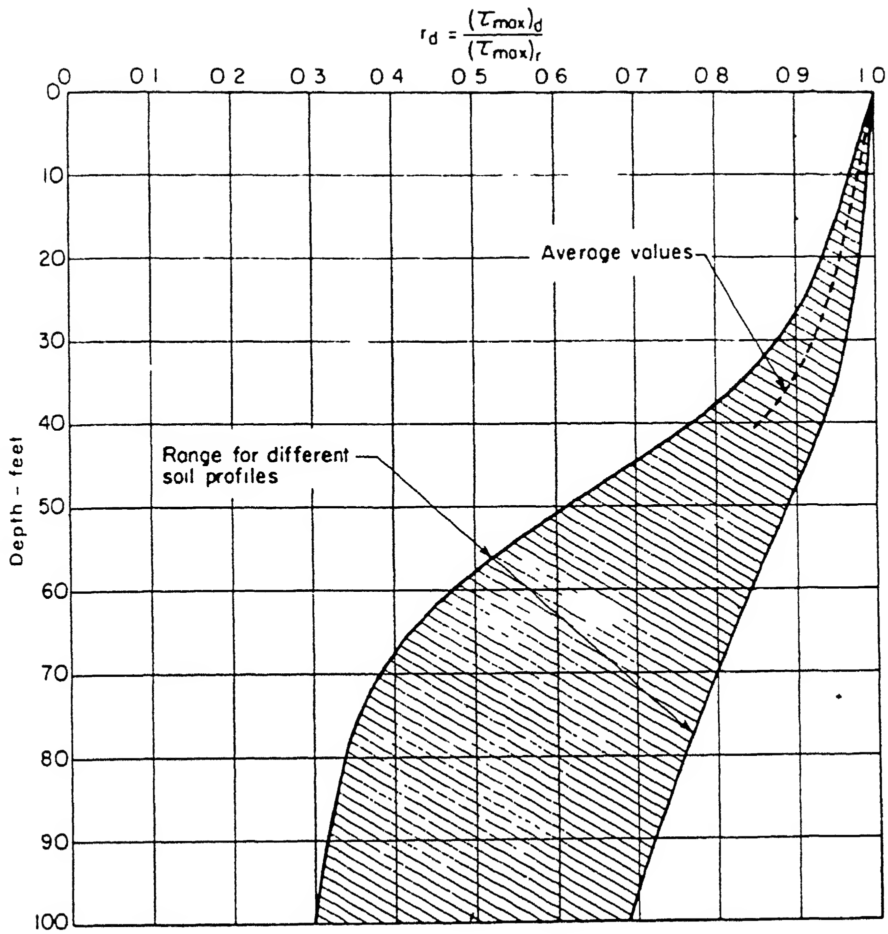

FIG – 2.2.1-RANGE OF VALUES OF r_d FOR DIFFERENT SOIL PROFILES

The appropriate number of significant stress cycles N_c will depend on the duration of the ground shaking and thus on the magnitude of the earthquake.

Representative number of stress cycles are:

earthquake magnitude	No of significant cycle N_c
≤ 7	10
7.5	20
8	30

TABLE-2.2.1

The use of these values together with stresses determined from equation (2.2.1) provide a simple procedure for evaluating the stresses induced at different depths during an earthquake for which the maximum ground surface acceleration is known.

Determination of the cyclic shear stresses causing liquefaction of a given soil in a given number of stress cycle may be made by means of appropriate laboratory test program using cyclic loading triaxial compression test. The result of a number of such investigation on soils with different grain size, represented by the mean grain size(D_{50}) and at a relative density of 50% are summarized in Figures-2.2.2 to 2.2.4.

The result of these tests are expressed in terms of stress ratio $\sigma_{dc}/2\ \sigma_a$ causing liquefaction in 10,20 and 30 cycles, where σ_{dc} is the cyclic deviator stress and σ_a is the initial ambient pressure under which the sample is consolidated.

The stresses required to cause liquefaction for sands at other relative densities may be estimated from the fact that for relative densities up to 80% the shear stress required to cause initial liquefaction is approximately proportional to relative density.

Now stress ratio τ/σ_o' causing liquefaction under field conditions may be related with $\sigma_{dc}/2\sigma_o'$ by relation

$$\left(\frac{\tau}{\sigma_o'}\right)_l = \left(\frac{\sigma_{dc}}{2\,\sigma_a}\right)_l \times C_r \qquad (2.2.5)$$

C_r is a correction factor to be applied to laboratory triaxial test data to obtain the stress condition causing liquefaction in the field. It is found that C_r vary with the relative density as shown in Fig-2.2.5. For a given soil at a relative density(D_r) the stress ratio causing liquefaction in the field may be estimated from

$$\left(\frac{\tau}{\sigma_o'}\right)_l = \left(\frac{\sigma_{dc}}{2\,\sigma_a}\right)_l \times C_r \times \frac{D_r}{50} \qquad (2.2.6)$$

The values of $(\sigma_{dc}/2\sigma_a)_{l50}$ corresponding to D_{50} are taken from Figs -2.1.2 to 2.1.4.

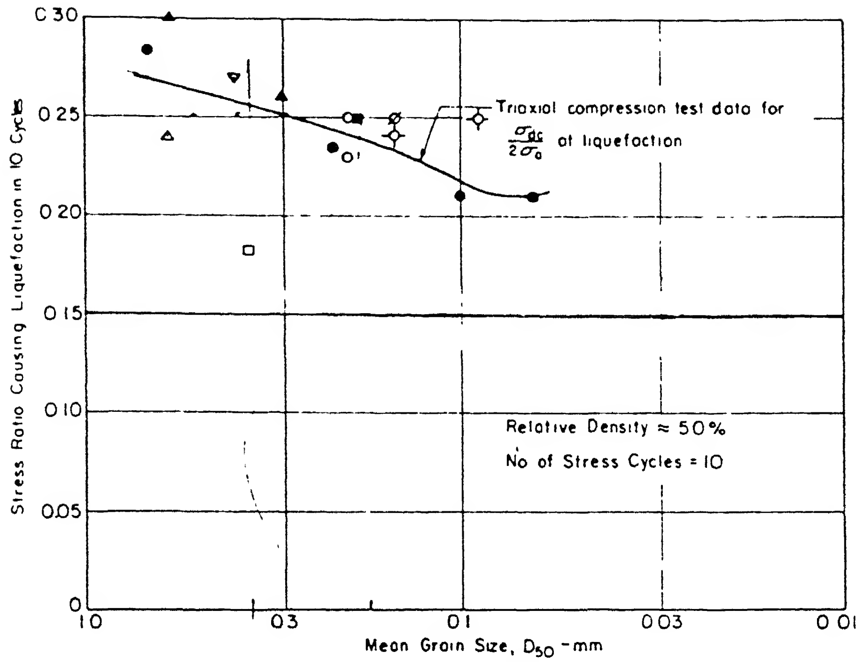

FIG - 2.2.2 .—STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS IN 10 CYCLES

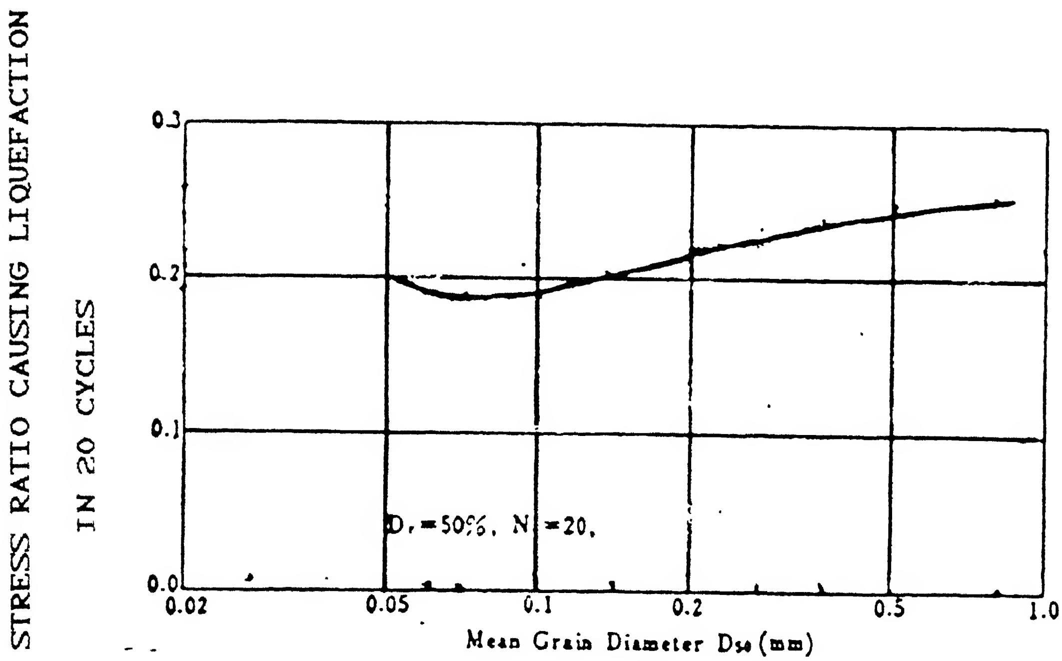

FIG - 2.2.3 STRESS CONDITIONS CAUSING LIQUEFACTION

IN 20 CYCLES

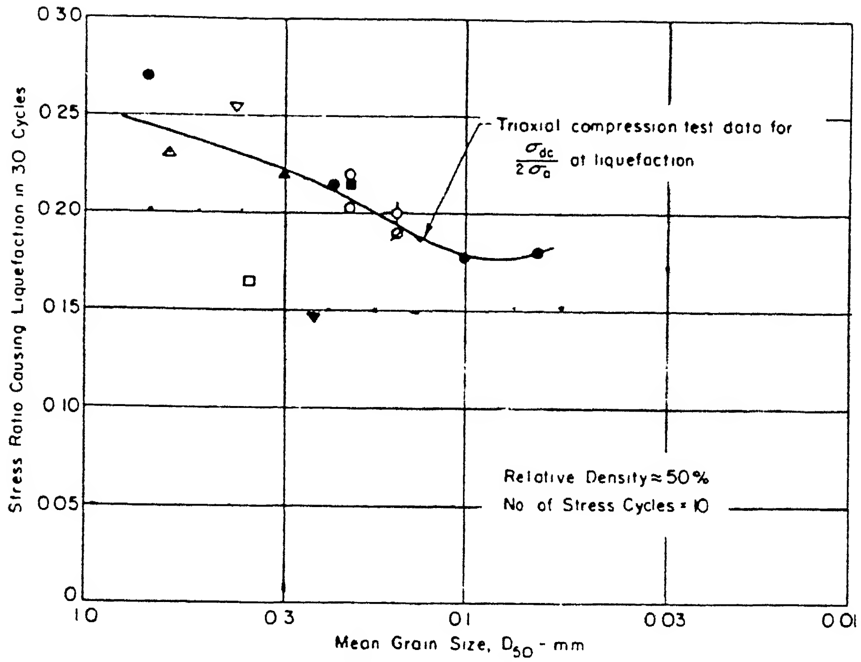

FIG - 2.2.4 .—STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS IN 30 CYLES

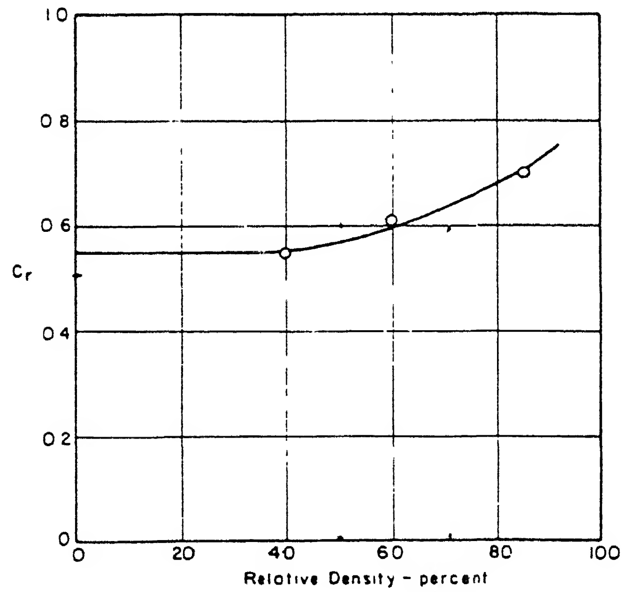

FIG - 2.2.5 -RELATIONSHIP BETWEEN c_{r} AND RELATIVE DENSITY

D_r can be found from the SPT N-value by the relation

$$D_r = \sqrt{\frac{N_1}{\sigma'_o + 70.0}} \qquad (\sigma'_o \text{ in kN/m}^2) \qquad (2.2.7)$$

Where N_1 is defined as $N_1 = C_n \times N$

$$C_n = 0.77 \times \log_{10}(20/\sigma'_v) \qquad (\sigma'_v \text{ is in Kgf/cm}^2)$$

In order to evaluate the liquefaction potential of a deposit it is necessary to determine whether the shear stress induced at any depth by the earthquake determined from equation (2.2.4) is sufficiently large to cause liquefaction or not at that depth as indicated by the equation (2.2.6).

Now Factor of safety can be found out from the relationship

$$F.S = \frac{\left[\frac{\tau}{\sigma'_o}\right]_{lD_r}}{\tau_{av}} \qquad (2.2.7)$$

If factor of safety is greater than 1.0 than soil will not liquefy.

2.3 Ishihara theory (1979):-

He reported that cyclic strength of sands containing fines depends not only on the amount of fines but also on its consistency characteristics. If the fines have a strong cohesion, it will inhibit separation of individual particles when the soil is about to liquefy. consequently, such a soil will exhibit a strong resistance to liquefaction. In contrast to this, if the fines content of materials with dry surface texture free from

adhesion, it will easily permit separation of individual particles and therefore the sand containing such fines will exhibit a large potential of liquefaction.

Thus it would appear likely that clay or silt sized particle such as rock flour having a low plasticity index will exhibit physical characteristics resembling those of cohesionless soils, and hence a high degree of potential to liquefaction.

He studied the effect on cyclic strength of the plasticity index of the fine grained soils and suggested that, it may be assumed that the part of the cyclic strength contributed by the presence of fines increases approximately in direct proportional to the plasticity index of the fines at a rate of $I_p/35$ for a sand containg about 30% fines. If this rate of increase in cyclic strength is assumed to hold approximately valid for any percentage of fines over 30%, the factor of $I_p/35$ may be incorporated into the empirical formula as a correction factor to allow for the effect of the consistency of fines. Thus he gave the equation of cyclic strength of soils as,

$$\left[\frac{\tau_{av}}{\sigma'_o}\right]_{20} = 0.009(N_1 + 13) + 1.67 \times 10^{-3} * I_p \times \log_{10} C \qquad (2.3.1)$$

where

N_1 is defined as $N_1 = C_n \times N$

N is the SPT value

$$C_n = 0.77 \log_{10} (20/\sigma'_v) \qquad (2.3.1.a)$$

I_p = Plasticity index

C = % of fines content

An equivalent cyclic stress ratio conceived to have developed in the field due to earthquake shaking is estimated from the following equation,

$$\frac{\tau_{av}}{\sigma'_v} = 0.65 \times \frac{a_{max}}{g} * \gamma_d \times \frac{\sigma_v}{\sigma'_v} \qquad (2.3.2)$$

Where $\gamma_d = 1.0 - 0.015 \times Z$ (2.3.2.a)

Z = depth in meter

After finding the values of the above equation factor of safety is found out,

$$F.S = \frac{\left(\frac{\tau_d}{\sigma'_o}\right)_{20}}{\left(\frac{\tau_{av}}{\sigma'_v}\right)} \qquad (2.3.3)$$

If factor of safety is greater than 1.0 than soil will not liquefy.

2.4 Tatsuoka et al. Theory (1980):-

They reviewed and compared several methods to evaluate cyclic undrained strength of sandy soils from standard penetration resistances and reported that empirical equations which do not take into account of the effects of grain size on N-value can be rather conservative for fine or silty sands. when the effects of grain size on N-values are adequately taken into account, strengths estimated from N-values are very close to the measured strengths of undisturbed samples.

In any analytical method of liquefaction potential evaluation, in-situ cyclic undrained strength or liquefaction strength should be evaluated.

Based on the data of cyclic undrained triaxial strength of undisturbed samples from young alluvial sandy deposit and uncompacted hydraulic fills, they proposed the following equation,

$$Rl = .88 \times \left[\sqrt{\frac{N}{\sigma'_v + 70}} - .258 \log_{10} \left(\frac{D_{50}}{.35} \right) \right]$$

$$\text{for } .04 \le D_{50} \le .6 \text{ mm} \qquad (2.4.1)$$

$$Rl = .88 \times \left[\sqrt{\frac{N}{\sigma'_v + 70}} - 0.0567 \right]$$

$$\text{for } .6 \le D_{50} \le 1.5 \text{ mm} \qquad (2.4.2)$$

In this case, Rl is defined for double axial strain of 5 or 6%..

Equation (2.4.1) means that N-value should be corrected at least both for σ'_v and D_{50}.

They emphasized that the equations are valid only for young alluvial sandy deposits and uncompacted hydraulic fills.

They used the following index to represent the degree of resistance of a soil element against liquefaction

$$F_1 = R / L \qquad (2.4.3)$$

In which F_1 is the liquefaction resistance factor, R is the resistance of the soil element and L is the cyclic load applied

to soil element. In this method, the effects of diffusion of excess pore pressure during earthquake are not directly accounted for.

The value of R can be related to R_1 as

$$R = C_1\ C_2\ C_3\ C_4\ C_5\ R1 \qquad (2.4.4)$$

C_1 is the correction factor for the difference of in-situ confining pressure from those in triaxial tests and has been proposed as $(1 + 2\ K_o)/ 3$ by Ishihara and Li (1972). C_2 is a correction factor for the difference between in situ "random" loading forms during earthquake motions and the sinsuoidal loading form in triaxial tests and has been proposed as 1/0.55 ~ 1/.07 with the average being 1.62 by Ishihara and Yasuda(1975). C_3 is that for the effects of soil disturbances in the process of soil sampling and handling. C_4 is for the effects of densification in the process of sampling and handling. C_5 is the correction for the multi-directional shaking which may be anticipated in actual in situ loading conditions. According to seed (1979) this factor is equal to 0.9. Using the values of C_1 through C_5 described above, equation (2.4 4) becomes

$$R = 2/3 \times 1.62 \times 1.0 \times .9 \times R1 = R1 \qquad (2.4.5)$$

for $K_o = 0.5$. This value of K_o can be considered adequate for uncompacted reclaimed lands and alluvial deposits.

The cyclic load L can be estimated from

$$L = \left(\frac{\tau}{\sigma'_v}\right)_{av} = 0.65 \times \frac{\alpha_{ax}}{g} \times \frac{\sigma_v}{\sigma'_v} \qquad (2.4.6)$$

In which $\left(\frac{\tau}{\sigma'_v}\right)_{av}$ is the maximum stress ratio during an earthquake motion. α_{max} is the estimated maximum acceleration on the ground surface in gals; g is the acceleration of gravity (980 gals). σ_v is the total overburden pressure; σ'_v is the effective overburden pressure; γ_d is the stress reduction coefficient with available coefficient with a value less than 1 (Seed et al.1979). Iwaski et al.(1978) has reported that it had been found by comprehensive series of response analysis of soil layers that γ_d is a function of depth, the predominant period of input motion (T_c) and natural period (T_g) of the site concerned. As a simple correction among γ_d, T_e,T_g or so was not established yet, it was considered reasonable to estimate γ_d for the simplified method from the average relationship as

$$\gamma_d = 1.0 - 0.015 \times z \quad (z: \text{depth in meter}) \qquad (2.4\ 7)$$

It is obvious that the parameter F_l is not enough to express the severeness of the liquefaction of a sand deposit. It was found by Iwaski et al.(1978) that severe damage to foundation was found only when the value of F_l was considerably less than 1.

2.5 Tokimatsu and Yoshimi Theory(1983):-

They reported that the following conditions seem to support the use of the SPT N-value to express directly the resistance of sands to liquefaction, if a more standardized SPT procedure is used or an appropriate correction is made for adjusting energy loss due to friction between cathead and rope.

(i) The SPT test is an in-situ test which reflects stress history and strain history effects, soil fabric, and horizontal effective stress,in addition to the combined effects of the relative density and the vertical stress. All of the above factors are known to influence the resistance of sands to liquefaction but are difficult to retain in most so called 'undisturbed samples'.

(ii)Numerous case histories of soil liquefaction during past earthquakes are available for which the SPT N-values before the earthquake are known. The method based on field performance with the SPT N-values can, therefore reflect in situ soil characteristics under real stress conditions during earthquakes which are difficult to be thoroughly simulated in the laboratory.Besides abundant SPT data in soils with high liquefaction potential will become useful for future earthquake.

They studied the effect of the fine content on the SPT N-values and on the resistance to liquefaction and concluded that, using the fine content as an index parameter for estimating the liquefaction resistance has the following advantages:

(i)The fine content is better correlated than the mean grain

size with the degree of damage due to soil liquefaction. (ii)The fine content is probably better related with soil consistency which in turn is related to undisturbed shear strength of soil. (iii) The fine content can be determined more easily than the mean grain size by washing the soil sample through a 75 micron sieve.

On the basis of the above discussion, he conducted a critical review of field performance of sandy soils deposits during past earthquakes in order to establish a more reliable empirical chart for estimating their liquefaction resistance.

In order to review case histories of soil liquefaction, a special attempt was made to re-examine shear stress developed during earthquakes, to asses the effect of the procedure of the SPT and soil types on the liquefaction resistance and to classify the degree of liquefaction.

Shear stress ratio to represent seismic ground motions:-

Based on the extensive laboratory test results of liquefaction of saturated sands, effect of seismic ground motions causing liquefaction may be represented by two quantities (i)horizontal ground accelerations and (ii)number of cycles of significant ground motions.

The finding is incorporated in the following equation for dynamic shear stress ratio for a given depth at a given site

$$\frac{\tau_d}{\sigma_o'} = \frac{\alpha_{max}}{g} \times \frac{\sigma_o}{\sigma_o'} \times \gamma_d \times \gamma_h$$

(2.5.1)

Where

τ_d - Amplitude of uniform shear stress cycles equivalent to actual seismic shear shear stress time history

α_{max}-- The maximum horizontal acceleration at ground surface

σ_o' - Initial effective vertical stress

σ_o - Initial vertical stress contributing to the shear stress defined by

$$\sigma_o = \int_o^z \gamma_t dz \qquad (2.5.1.a)$$

γ_t - Unit weight of soil

z - Depth below the ground surface (in meter)

γ_d and γ_n are correction factors in terms of depth

M - Earthquake magnitude

γ_d - $1-0.015\times Z$ (Z is in meter)

γ_n - $0.1(M-1)$

SPT N - value to express liquefaction resistance:-

They used the questionnaire survey conducted by yoshmi et al.(1983) showing that the following three procedures to drop the hammer onto rod during the SPT measurement are frequently used in current Japanese practice. (i) the trip monkey method (ii) the cathead and rope method with two turns of rope around the cathead. (iii) the same method as (ii) except for a manner of

releasing the rope, in that the rope is completing thrown off the cathead in order to reduce rope friction.

They showed that SPT N-value by the cathead and rope method is greater by 20% than trip monkey method for any N-values up to 40.

So, $N_{cj} = 1.2\ N_{tj}$ (2.5.2)

cj – means cathead and rope method and tj means the trip monkey method in japan.

$N_{cj} = 1.4 \times N_{tj}$ (2.5.3)

cf – means the cathead and rope method of foreign countries.

Since it is well known that SPT N-values are influenced by the effective confining pressure in the soil as well as the soil density which may reflect the undrained strength of soil; corrected SPT N-values for a reference confining pressure have occasionally been used for practical purposes.

The SPT N-values normalized for σ'_o = 1 Kgf/cm^2; N_1; which is adopted may be approximately given by

$$N_1 = C_n \times N = \frac{1.7}{\sigma'_o + .7} \times N \qquad (2.5.4)$$

C_n is a function of the effective vertical stress, σ'_o in Kgf/cm^2 at the time when and at the depth where the penetration test was conducted.

The above equation is based on a simplified relation by Meyerhof (1957) which in turn was based on the test results by Gibbs and Holtz (1957) for the effective vertical stress up to 2.8 Kgf/cm^2 and is also equivalent to that recently proposed by Seed (1979) for $\sigma'_o \leq 1.5$ Kgf/cm^2 which may cover the range of general interest for soil liquefaction.

To express resistance of soil liquefaction, laboratory tests were conducted on undisturbed samples to determine the soil resistance in terms of stress ratio and a strain level and its relation to SPT N-values. Because of great difficulty in obtaining high quality undisturbed samples of dense sand, the method based on laboratory testing procedure on so called "undisturbed samples" tends to underestimate the resistance to liquefaction , and the degree of underestimation could increase as the sand become denser.

SO a large amount of undisturbed samples of dense sand was obtained by means of an in-situ freezing method considered to be most suitable for obtaining high quality undisturbed samples of clean sands and hence can yield a most appropriate value of soil strength.

They proposed a relationship between shear stress ratio and relative density for these soils in terms of axial strain amplitude at the end of 15 cycles. The relationship is represented by the following equation.

$$\frac{\sigma'_o}{2 \times \sigma'_c} = a \left[\frac{D_r}{100} + \left(\frac{D_r}{C} \right)^n \right] \tag{2.5.5}$$

In which a and n are empirical constants. C is an empirical parameter depending on strain amplitude.

The term C is added in order to take into account the effect of strain.

Because the relationship between C and strain for reconstituted samples is reasonably linear on a semi-log chart with a = 0.45 and n = 14

$$C_a = 97 - 19 \log DA \text{ (for triaxial test)} \tag{2.5.6}$$

$$C_s = 94 - 19 \log \gamma \text{ (for simple shear test)} \tag{2.5.7}$$

In which DA is double amplitude of axial strain and γ is single amplitude of shear strain.

They showed that equation (2.5.5) together with equation (2.5.6) could simulate reasonably well the relationship between stress ratio an relative density of the undisturbed clean sands.

On the other hand, a relationship between relative density and N-value for sands without fines was given by Meyerhof(1957)

$$D_r = 21.0 \times \sqrt{\frac{N}{\sigma' + 0.7}} \tag{2.5.8}$$

In which σ'_0 denotes the effective overburden stress in Kgf/cm^2.

A drastic decrease in SPT N-values for saturated sands with fines was observed in the field case histories. The relationship for this fine sand may be represented by the following equation assuming that ΔN_f =15, although the general applicability of

this equation for estimating the relative densities of fine sands is yet to be proven.

$$D_r = 21\sqrt{\frac{N}{\sigma'_o + 0.7} + \frac{\Delta N_f}{1.7}} \qquad (2.5.9)$$

$$D_r = 16\sqrt{N_1 + \Delta N_f} \qquad (2.5.10)$$

In which ΔN_f is a constant and can be considered as a correction term for taking into account the effect of fines content.

Assuming that the relationship between shear stress ratio and relative density is uniquely defined by equation (2.5.5) irrespective of the presence of fines, the following relationship can be obtained by substituting the equation (2.5.10) into equation (2.5.5)

$$\frac{\sigma_{do}}{2\sigma'_o} = a\left[\frac{16\sqrt{N_1 + \Delta N_f}}{100} + \left(\frac{16\sqrt{N_1 + \Delta N_f}}{C_a}\right)^n\right]$$

(2.5.11)

The equation(2.5.11) can represent directly the relationship between shear stress and SPT N-values in terms of shear strain.

The following considerations must be given when one attempts to relate the undrained strength of in-situ saturated sands during an earthquake and that obtained by the cyclic triaxial test even if the soil sample is perfectly undisturbed (i) the effect of system compliance including membrane penetration and of

end friction (ii) the difference between the triaxial condition and the sample shear condition (iii) the effect of multidirectional shear.

However, for lack of direct evaluation of the above using high quality undisturbed samples of dense sand, the correction factor C_r =0.57 proposed by De Alba et al.(1976) based on the large scale simple shear tests on a reconstituted sand will be adopted here

$$\left(\frac{\tau_L}{\sigma'_o}\right)_{field} = C_r \times \left(\frac{\sigma'_c}{2 \times \sigma'_c}\right)_{triaxial} \quad (2.5.12)$$

Thus the relationship between dynamic shear stress ratio and SPT N-values with respect to shear strain amplitude for representative samples of in-situ soils may be defined by the following equation.

$$\frac{\tau_L}{\sigma'_o} = a\, C_r \left[\frac{16\sqrt{N_1 + \Delta N_f}}{100} + \left(\frac{16\sqrt{N_1 + \Delta N_f}}{C_s}\right)^n\right] \quad (2.5.13)$$

where a=0.45, C_r =0.57, n=14, ΔN_f =0.0 for clean sands and ΔN_f = 5 for silty sands are assumed.

In order to eliminate ΔN_f in equation (2.5.13) an adjusted SPT N-value is defined by $N_a = N_1 + \Delta N_f$ in which ΔN_f is a correction

term defined in Table-2.5.1.

Correction factors for determining SPT N_a value

Fines content(Fc)	ΔN_f
0-5	0
5 - 10	interpolae
10 -	0.1×Fc +4

TABLE-2.5.1

thus final equation will be

$$\frac{\tau_l}{\sigma'_o} = a \times C_r \times \left[\frac{16\sqrt{N_a}}{100} + \left\{ \frac{16\sqrt{N_a}}{C_s} \right\}^n \right] \qquad (2.5.14)$$

Factor of safety against liquefaction, F_l is usually defined in terms of shear stress ratio by the following equation

$$F_l = \frac{\left[\tau_l / \sigma'_o \right] \text{ at a given strain}}{\tau_d / \sigma'_o} \qquad (2.5.15)$$

Because it is conceivable that the criterion presented here in is based on SPT N-value determined by the free-fall method in which ratio of net impact energy delivered to the rod is probably close to 0.8; appropriate correction should be made when using these criterion with SPT N-value determined by other methods.

On the basis of a review of field behaviour during several

earthquake together with laboratory test on high quality undisturbed samples of sand, the following conclusion may be drawn :- (i) sands containing more than 10%fines have much greater liquefaction resistance than clean sands having the same SPT N-values.

(ii) extensive damage due to liquefaction would not occur for clean sands with SPT N-values greater than 25, and silty sands containing more than 10% fines whose SPT N-values are greater than 20.

(iii) sands containing gravel particles seem to have less resistance to liquefaction than clean sands without gravel having the same SPT N-value.

(IV) Soil containing more than 20% clay would hardly liquefy unless their plasticity index are low.

2.6 Shibata and Teparaksa Theory (1988):-

They reviewed field behaviour during several recent earthquake with emphasis on the cone penetration test (CPT)$q_c$values and the grain size of the soils. The field correlation between the earthquake induced cyclic stress ratio and normalized cone resistances indicate that fine grained soils with a mean grain size of D_{50}< 0.25 mm have grater resistance to liquefaction than do clean sands (D_{50}≥ 0.25 mm) having the same $q_c$values.

The SPT has been widely used for many years, but the CPT has now become more popular as an in-situ test for site investigation and geotechnical design. The most important advantages of the CPT are its simplicity, repeatability, accuracy and continuous record.

They used two parameter, the normalized cone resistance and the cyclic stress ratio generated by earthquake ground motion.

Normalized cone resistance, q_{c1}:- Because the CPT $q_c$value are influenced by the effective confining pressure as well as by the soil density, for practical purpose,corrected qc value for a reference effective overburden pressure, σ'_o have been used. The corrected cone penetration resistance value(q_{c1}) is obtained from the observed cone value(q_c) from the following expression,

$$q_{c1} = C_1 \times q_c = \left[\frac{0.17}{\sigma'_o + 0.7}\right] q_c \quad \text{(Mpa)} \qquad (2.6.1)$$

In which C_1 is a function of σ'_o, at that depth at which the test was conducted.

Cyclic stress ratio, τ/σ'_o :- The cycle stress ratio that develops in the field during an earthquake is estimated from the following equation.

$$\frac{\tau}{\sigma'_o} = 0.1\ (M-1)\ \frac{\alpha_{max}}{g} \times \frac{\sigma_o}{\sigma'_o} \times (1-0.015 \times z) \qquad (2.6.2)$$

where

τ - the amplitude of uniform shear stress cycles equivalent

to actual seismic shear stress time history

σ_o – the initial vertical stress

σ_o' – initial vertical effective stress

M – magnitude of earthquake

α_{max} – maximum horizontal acceleration at ground surface

They studied the relation between q_{c1} value and mean grain size, D_{50} for all the liquefied soil and reported that there is fairly well defined trend in which upper bound of the q_{c1} values for liquefied soil decreases with the decrease in mean grain size when D_{50} <0.25mm ; this shows that the smaller the grain size, the greater the liquefaction resistance. On the other hand, for clean sand with a value of D_{50} ≥0.25mm, the upper bound of q_{c1} values for liquefied soil seems to be independent of grain size.

They defined the critical value of CPT, $(q_{c1})_{cr}$, that separates liquefaction and non-liquefaction. Since the critical boundary to define the liquefaction potential for sand with D_{50}≥0.25mm is assumed to be independent of grain size, the $(q_{c1})_{cr}$ value may be uniquely determined by τ/σ_o' regardless of grain size.

Thus, the critical value $(q_{c1})_{cr}$, for soils with D_{50}≥0.25mm can be represented by the function of τ/σ_o'

$$(q_{c1})_{cr} = f(\tau/\sigma'_o) \qquad (2.6.3)$$

In contrast, for fine grained soil with D_{50}<0.25mm, the boundary line are straight with a slope of 1.0 on a log-log graph. Thus, $(q_{c1})_{cr}$ value increase in proportion to D_{50} and the functional form is

$$(q_{c1})_{cr} = C_2 \times f(\tau/\sigma'_o) = \frac{D_{50}}{0.25} \times f(\tau/\sigma'_o) \qquad (2.6.4)$$

they plotted $\dfrac{(q_{c1})_{cr}}{C_1}$ verses (τ/σ'_o)

and using the nature of the above curve as hyperbolic suggested the following expression

$$\frac{(q_{c1})_{cr}}{C_2} = f(\tau/\sigma'_o) = \frac{b\left[\tau/\sigma'_o - a\right]}{b \times c + \left[\tau/\sigma'_o - a\right]} \qquad (2.6.5)$$

In which C_2= 1.0 for soils with $D_{50} \geq$ 0.25mm and

$C_2 = D_{50}/0.25$ mm for soils with D_{50}< 0.25 mm

The parameter a=intersection of the boundary curve with the τ/σ'_o axis, b = the value of$(q_{c1})_{cr}/C_2$ when τ/σ'_o becomes infinite and C = initial slope of the the hyperbolic curve on τ/σ'_o axis.

they assumed that a = 0.06, and b = 25 Mpa and

C =6.4 × 10^{-3} 1/Mpa

we will obtain the equation as

$$(q_{c1})_{cr} = C_2\left[5 + 20\left(\frac{\tau/\sigma'_o - 0.1}{\tau/\sigma'_o + 0.1}\right)\right] \text{ Mpa} \qquad (2.6.6)$$

And the curve between $(q_{c1})_{cr}/C_2$ verses τ/σ'_o represent the boundary between liquefaction and no liquefaction.

Now substituting τ/σ'_o from equation (2.6.2) in above equation we will know $(q_{c1})_{cr}$. $(q_{c1})_{cr}$ is corrected using equation (2.6.1) to obtain critical cone resistance $(q_c)_{cr}$.

This $(q_{c1})_{cr}$ is compared with known q_{c1} value and the zone of liquefaction where $(q_{c1})_{cr}$ is more than q_{c1} is found.
The factor of safety against liquefaction is evaluated as

$$F.S = \frac{q_{c1}}{(q_c)_{cr}} \qquad (2.6.7)$$

2.7 Chang method (1989):-

The possibilities of predicting liquefaction potential of saturated soil by standard penetration test (SPT), and static cone penetration test (SCPT) are mainly based on the relations between N (SPT blow count), qc (static point resistance) and relative density of soil. The test results show that the liquefaction resistance of soil is not alike if soil fabric or cause of formation is different, even if density of soil or N is alike.

To a certain degree, rigidity of soil is more suitable than density of soil in describing liquefaction resistance of soil, it is feasible to use wave velocity test of soil as a means to find liquefaction of soil.

The relative curves of sandy soil and sandy loam (V_s − N, qc − N) are acquired respectively by way of analysing the test data and then experimental formulas of the curves are also obtained as follows:

For sandy soil,

$$V_s = \frac{N}{0.004 \times N + 0.003} \qquad (2.7.1)$$

$$qc = 6.025 \times N + 12.52 \qquad (2.7.2)$$

For sandy loam,

$$V_s = \frac{N}{0.0058 \times N + 0.004} \qquad (2.7.3)$$

$$qc = 3.815 \times N + 10.81 \qquad (2.7.4)$$

Where N = SPT value

Field studies of liquefaction in china since 1966, has resulted in the development of an empirical criterion for evaluating the liquefaction potential of soil in the field, based on the results of the SPT, recently, the new criterion is given by the equation (new earthquake proof code, China)

$$N_{crit} = \overline{N} \left(1 + 0.1 \, (ds \; 3 \,) - 0.1 \, (dw - 2 \,) \right) \sqrt{3/pc} \qquad (2.7.5)$$

where ds = depth to sand or sandy loam layer under consideration.

dw = depth of water table below ground surface

pc = clay content %

$\overline{N}$ = critical SPT blow count of the liquefied discriminate, function of the shaking intensity or acceleration in gravity units.

intensity	$\overline{N}$ (near field earthquake)	$\overline{N}$(far field earthquake)
≤7	6	8
8	10	12
9	16	

TABLE-2.7.1

In the formulas (2.7.1) to (2.7.4) N is replaced by $\overline{N}$, when critical wave velocity of soil $\overline{V}_s$ and critical point resistance of soil $\overline{qc}$ are obtained as follows:

For sandy soil

$$\overline{V}_s = \frac{\overline{N}\ (0.9 + 0.1(ds - dw))}{0.004\ N\ (0.9 + 0.1(ds - dw)) + 0.003} \quad \text{...(2.7.6)}$$

$$\overline{qc} = 6.025\ \overline{N}(0.9 + 0.1(ds - dw)) +12.52 \quad (2.7.7)$$

for sandy loam

$$\overline{V}_s = \frac{\overline{N}\ (0.9 + 0.1\ (ds - dw))\sqrt{3/pc}}{0.0058\ \overline{N}\ (0.9 + 0.1(ds - dw))\sqrt{3/pc} + 0.004} \quad (2.7.8)$$

$$\overline{qc} = 3.815\times\ (0.9 + 0.1(ds - dw))\sqrt{3/pc} + 10.81 \quad (2.7.9)$$

After finding the value of these quantities we can say that the site will be non-liquefied if the $V_s > \overline{V}_s$ and $qc > \overline{qc}$

CHAPTER 3

GENERAL STRATA CONDITIONS OF THE CHOSEN SITES IN UTTAR PRADESH FOR LIQUEFACTION POTENTIAL EVALUATION

3.1 General The sites that have been selected for studying their liquefaction potential are Gorakhpur Sadar, Deoria Sadar, Kanpur city, Anola, Shahjahnpur, Dalmau in Raebareli. Selection of these sites are based on the fact that these are earthquake prone and lies in different earthquake zones as per IS Code 1893-1984.

The other reason being the nature of soil deposit which is likely to liquefy during earthquakes. The hazard associated with the soil liquefaction during earthquake is well known in such deposits consisting of fine to medium sands and sands containing low plasticity fines. the site conditions for the places under consideration are briefly presented in the following section. From the soil investigation carried out by I.I.T Kanpur and other government and private agencies, for various sites of the different area as mentioned below, data have been collected for the present study. The same have been analysed and necessary input data for the developed program have been prepared by averaging process. The location of various sites is shown in a map (Fig-3.1). A table is presented to give the number of borehole logs collected and analysed for different sites.

FIG - 3.1 SITE LOCATIONS IN THE MAP OF UTTAR PRADESH

STATISTICS OF BORE HOLE

AREA	BORE HOLE LOCATIONS	NUMBER OF BORE HOLES	DEPTH OF BORE-HOLES (meter)
GORAKHPUR	ALINAGAR	2	20
	BASARARPUR	2	20
	KHOONIPUR	1	20
		2	10
	NICHAUL	2	10
		1	20
	CHARUCHANDRAPURI	1	20
		3	15
		2	20
DEORIA	PATHARDEVA	2	20
		1	15
	SALEMPUR	3	20
	BHATPARRANI	1	20
		2	10
KANPUR	I.I.T	2	30
		1	10
	ACHARAYA NAGAR	4	15
		1	20
	O.F.C	3	3
		2	25
	H.B.T.I	2	10
		1	20
	CANTOMENT	2	10
		1	20
	PANKI	4	10
	RAWATPUR	2	20
		2	10
	ARAMAPUR	2	6
		3	8
	POLICE LINE	2	10
		1	20
	CHAUBEYPUR	2	10

cont.

KANPUR	SAROJINI NAGAR	2 2	30 15
SHAHJAJHNPUR	---	6 14	30 22
ANOLA	----	3	30
RAIBARELY	---	4	20

TABLE-3.1

The horizontal ground acceleration that a site is likely to experience during earthquake has been computed by using available semi-empirical correlation for such deposits and has also been presented in the this chapter.

3.2 Site conditions

3.2.1 Gorakhpur and Deoria Sadar The alluvial tract between the great Gandak and Ghagra rivers forms part of the great Ganga basin. The areas covered greater part of Gorakhpur and Deoria districts of Uttar Pradesh and a small part of Champarn district, Bihar. Lying about 15Km south of the foot hills of the Siwalik ranges of Nepal, the area extends from Indo-Nepal border on the north to the Ghagra river in the south with a gentle southerly slope. Occurrence of sand ridges and mounds here and there breaks the monotony of an otherwise flat alluvial plain. The area is underlain by the quaternary alluvial deposits by the rivers. Average D_{50} vary from 0.078 to 0.19mm. Top layer is silty clay followed by silty sand. In some places in Gorakhpur organic clay is also present at depth of 6 to 8m. The seasonal water table depth fluctuates in between 2.5 to 10m from the ground level. The variation in plasticity index is quite large.

According to IS Code 1893-1984 it lies in earthquake zone IV. Being nearby to the geologically active zone of Nepal Himalayas, heavy to moderate earthquake can occur in this area. This is also evident from the seismological map as shown in Fig- 3.2.

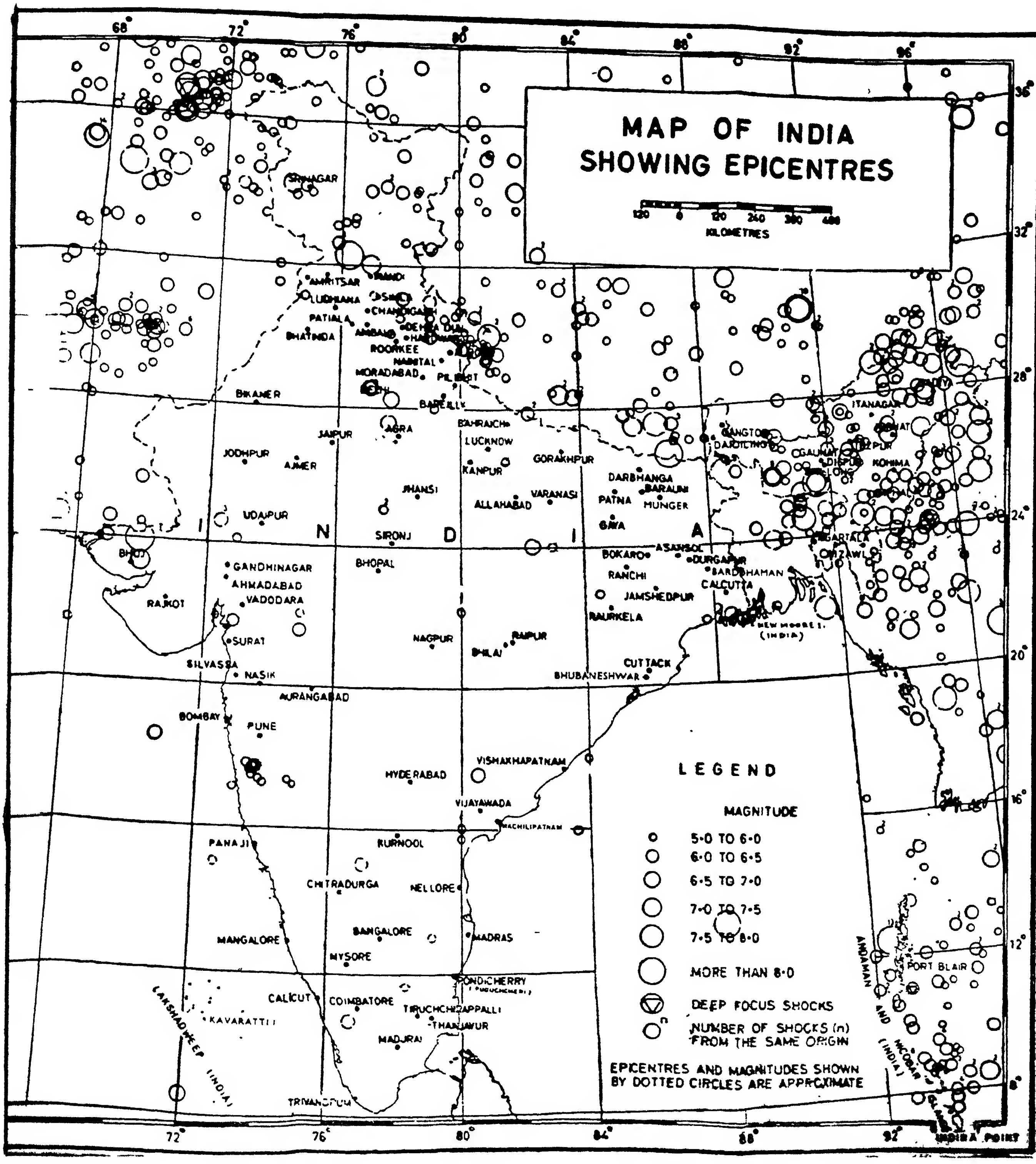
MAP OF INDIA
SHOWING EPICENTRES
KILOMETRES
LEGEND
MAGNITUDE
5·0 TO 6·0
6·0 TO 6·5
6·5 TO 7·0
7·0 TO 7·5
7·5 TO 8·0
MORE THAN 8·0
DEEP FOCUS SHOCKS
NUMBER OF SHOCKS (n) FROM THE SAME ORIGIN
EPICENTRES AND MAGNITUDES SHOWN BY DOTTED CIRCLES ARE APPROXIMATE
SRINAGAR
AMRITSAR
LUDHIANA
SHIMLA
CHANDIGARH
PATIALA
BHATINDA
AMBALA
ROORKEE
MORADABAD
PILIBHIT
BAREILLY
BIKANER
JAIPUR
AGRA
BAHRAICH
LUCKNOW
JODHPUR
AJMER
KANPUR
GORAKHPUR
JHANSI
VARANASI
ALLAHABAD
DARBHANGA
PATNA
MUNGER
GAYA
UDAIPUR
SIRONJ
I N D I A
BHUJ
GANDHINAGAR
AHMADABAD
VADODARA
RAJKOT
BHOPAL
BOKARO
ASANSOL
DURGAPUR
BARDDHAMAN
RANCHI
CALCUTTA
JAMSHEDPUR
RAURKELA
SURAT
NAGPUR
RAIPUR
BHILAI
CUTTACK
BHUBANESHWAR
SILVASSA
NASIK
AURANGABAD
BOMBAY
PUNE
VISHAKHAPATNAM
HYDERABAD
VIJAYAWADA
MACHILIPATNAM
PANAJI
KURNOOL
CHITRADURGA
NELLORE
BANGALORE
MADRAS
MANGALORE
MYSORE
PONDICHERRY
CALICUT
COIMBATORE
TIRUCHCHIRAPPALLI
THANJAVUR
KAVARATTI
MADURAI
LAKSHADWEEP (INDIA)
TRIVANDRUM
GANGTOK
DARJILING
ITANAGAR
GAUHATI
SHILLONG
DISPUR
KOHIMA
AGARTALA
AIZAWL
NEW MOORE I. (INDIA)
PORT BLAIR
ANDAMAN AND NICOBAR (INDIA)
INDIRA POINT
68°
72°
76°
80°
84°
88°
92°
96°
36°
32°
28°
24°
20°
16°
12°
8°

FIG - 3.2

The typical bore logs from Nichaul, in Gorakhpur Sadar are shown in Fig-3.3. For the various bore log data collected, the variation of the average SPT value with depth is shown in Fig-3.4.

The typical bore logs from Pathardeva, in Deoria Sadar are shown in Fig-3.5. For the various bore log data collected, the variation of the average SPT value with depth is shown in Fig-3.6.

3.2.2 Anola This forms a part of the alluvium plain of the Ganga basin. The deposits of this tract belong, so to say, to the last chapter of earth's history and conceal beneath them the northern fringe of the Peninsular formation and southern fringe of the extra-Peninsular formation.

The area being Gangetic alluvium plain comprises of various grades of sand, silt, clay and kankar. Characteristics of the sub-soil changes widely both in horizontal as well as vertical direction.

The top stratum is highly desiccated, very hard greyish brown silty clay. It is generally located between 0-1.5 below the ground level. This consists of about 30% clay and 70% silt. Its average N-value is 6-12. Plasticity index is 10 to 15.

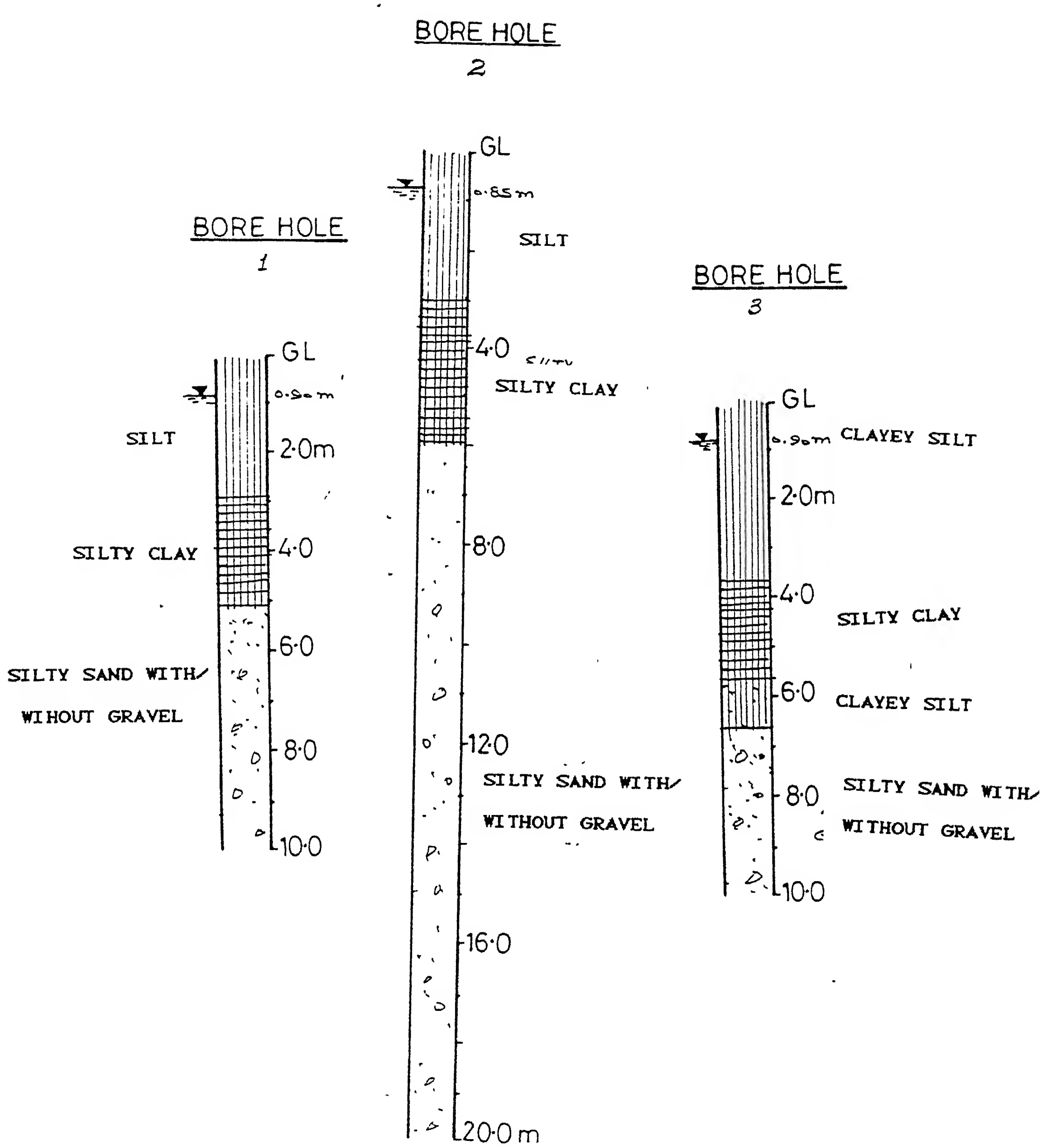

FIG - 3.3 BORELOGS: NICHAUL IN GORAKHPUR SADAR

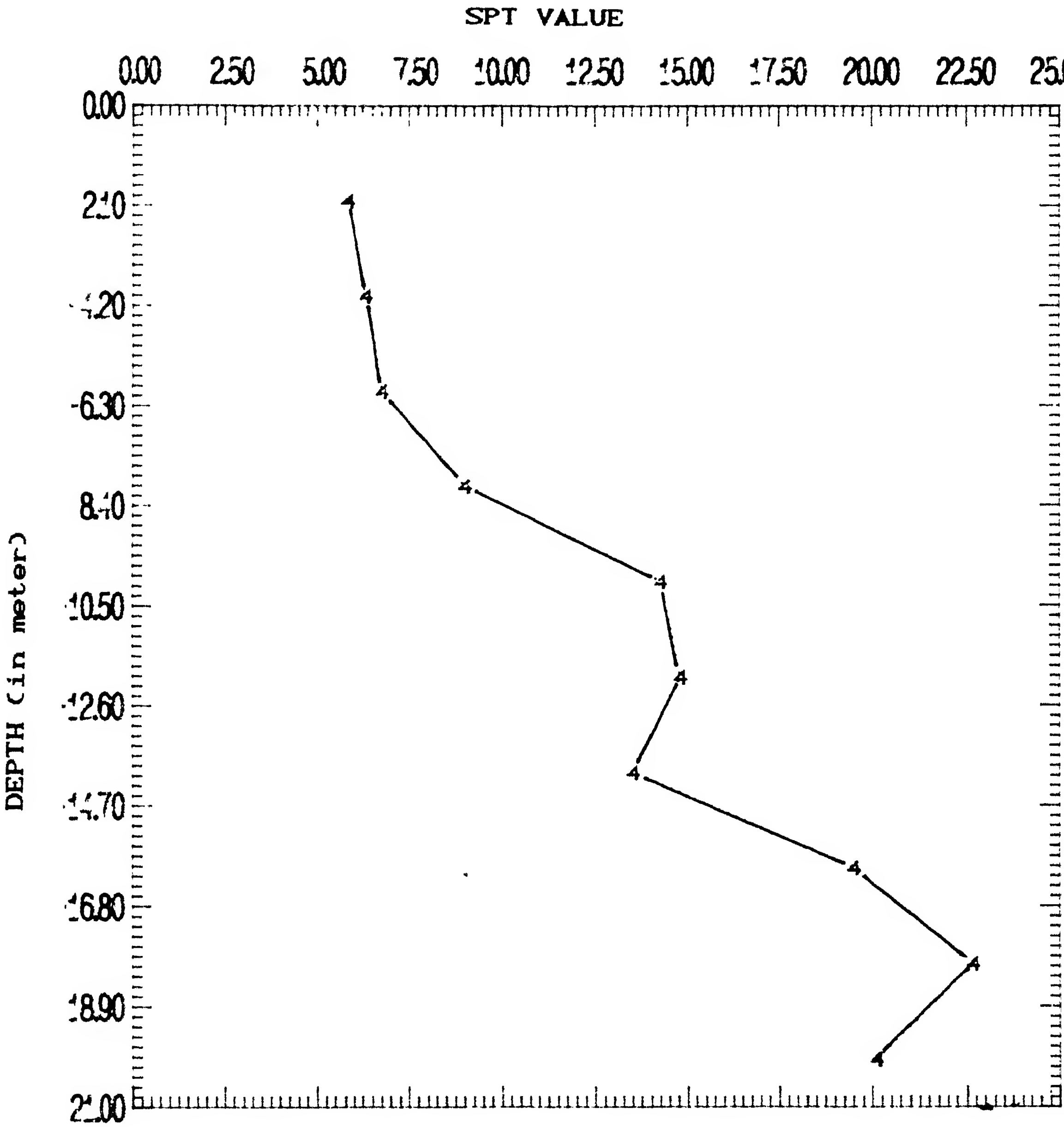

FIG - 3.4 **VARIATION OF SPT VALUE WITH DEPTH (GORAKHPUR)**

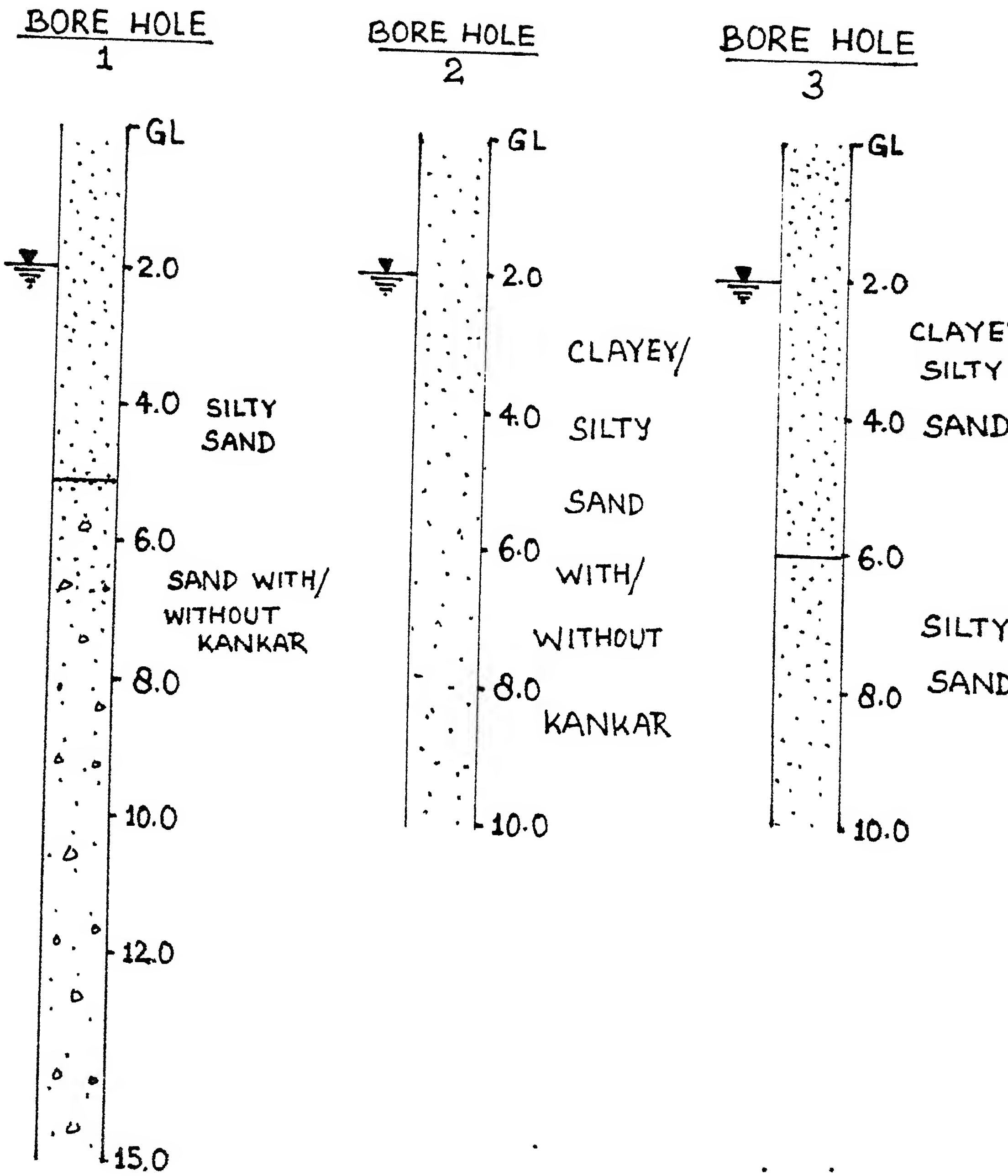

FIG - 3.5 BORELOGS: PATHERDEWA IN DEORIA SADAR

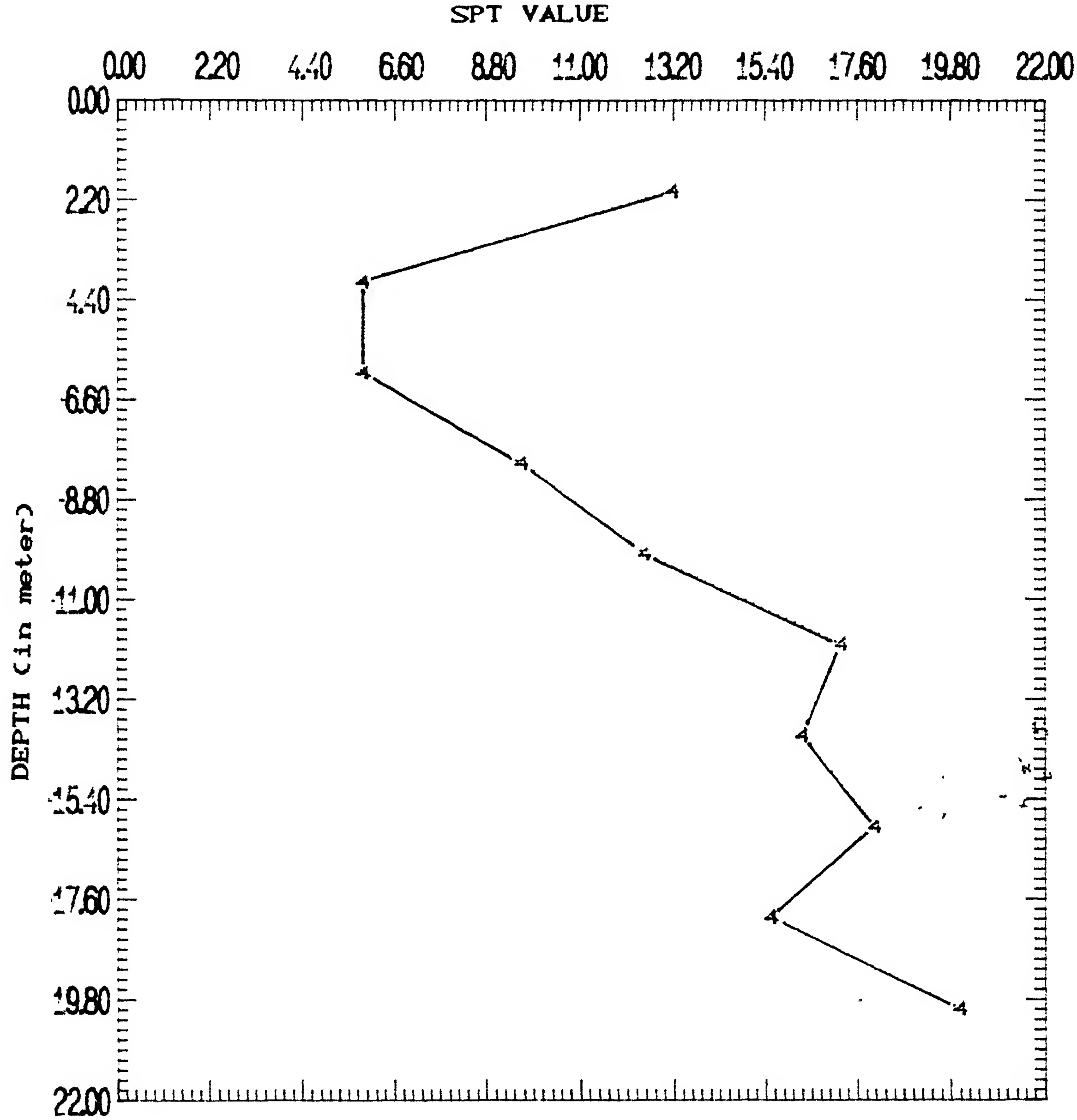

FIG - 3.6 **VARIATION OF SPT VALUE WITH DEPTH (DEORIA)**

Below the top stratum greyish brown clayey silt has been encountered between 1.5 to 3m below the ground level. This is mainly silt in nature. This is in very loose to loose state. N-value in this stratum varies from 4-12.

Below this is predominantly fine grained uniformly graded loose to medium dense sand deposit up to 15m depth. From 15 to 24m depth there is blackish grey silty soil. From 24 to 30m depth soil layer is composed of blackish grey fine sand. The average ground water table is at shallow depth (with in 2m) from the ground level. Plasticity index of the soil underlying the top layer is generally 0 to 2%. Average unit weight and D_{50} are $18kN/m^3$ and 0.154mm respectively. As for the seismic zoning classification of IS Code 1893-1984, it is situated in the border line of zone III and zone IV. This indicates that moderate to heavy damage is likely to occur during earthquake. This is also evident from the seismological map as shown in Fig-3.2. According to Negi and Ermenko(1968) the following tectonic units can be recognized in the region-:(i) frontal folded area (ii) areas of Delhi folding (iii) Lahore-Delhi ridge (iv) Punjab shelf (v) Delhi-Haridwar ridge (vi) West Uttar Pradesh shelf.

The typical bore log in Anola is shown in Fig-3.7. From the various bore log data collected, the variation of the average SPT value with depth is shown in Fig-3.8.

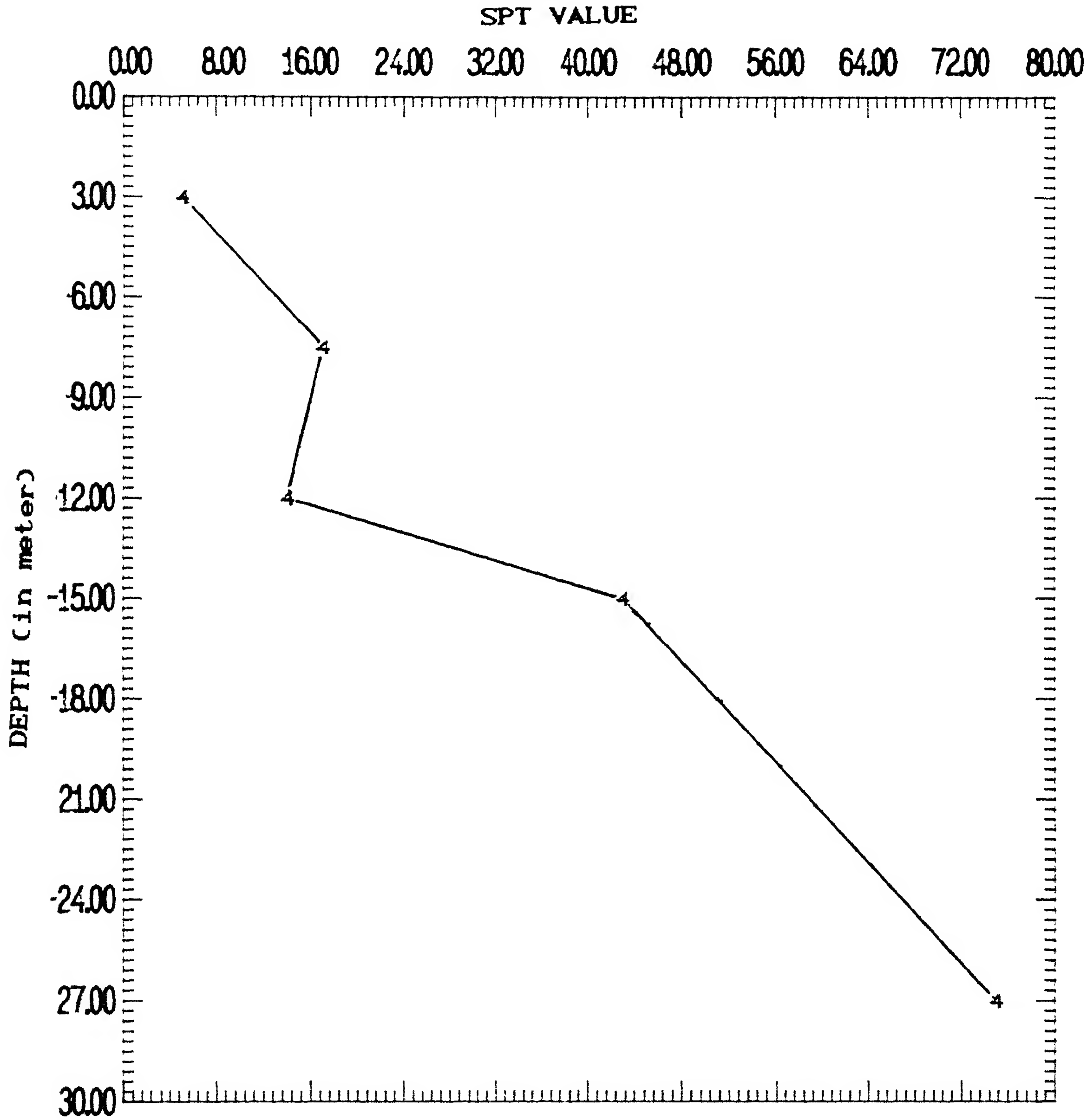

FIG - 3.8 **VARIATION OF SPT VALUE WITH DEPTH (ANOLA)**

3.2.3 Shahjahnpur This area also forms a part of the alluvium plain of the Ganga basin. Alluvium comprises of various grades of sand, silt, clay and kankar. General sub-soil stratification consists of top silty clay layer of 2 to 3m thickness underlain by mainly fine sand of uniform grain size with occasional intersections of silty clay and silty sand layers. The seasonal water table depth fluctuates in between 1 to 10m. The sand deposit is in a loose to medium dense state and below this level it is in dense to very dense state. Average D_{50} and unit weight are 0.872mm and 17kN/m^3 respectively. The clay content in top layer (1-3m) is 10 to 15%, whereas below the top layer clay content is in the order of 1 to 3%.

As per IS Code 1893-1984 it lies in zone III indicating moderate to heavy damages during earthquake. It is evident from the seismological map as shown in Fig-3.2. The significant tectonic belts present in the area around the site are same as in to Anola as mentioned above in sectoin 3.2.2.

The typical bore log in Shahjahnpur is shown in Fig-3.9. From the various bore log data collected, the variation of the average SPT value with depth is shown in Fig-3.10.

3.2.4 Raebareli This is also the part of alluvium plain of the Ganga basin. Filled up soil is found in the top layer. Below the top layer, soil is low to intermediately plastic in nature consisting of sandy silt of very low plasticity(ML group), clayey silt of very low plasticity(ML-CL group), clayey silt of

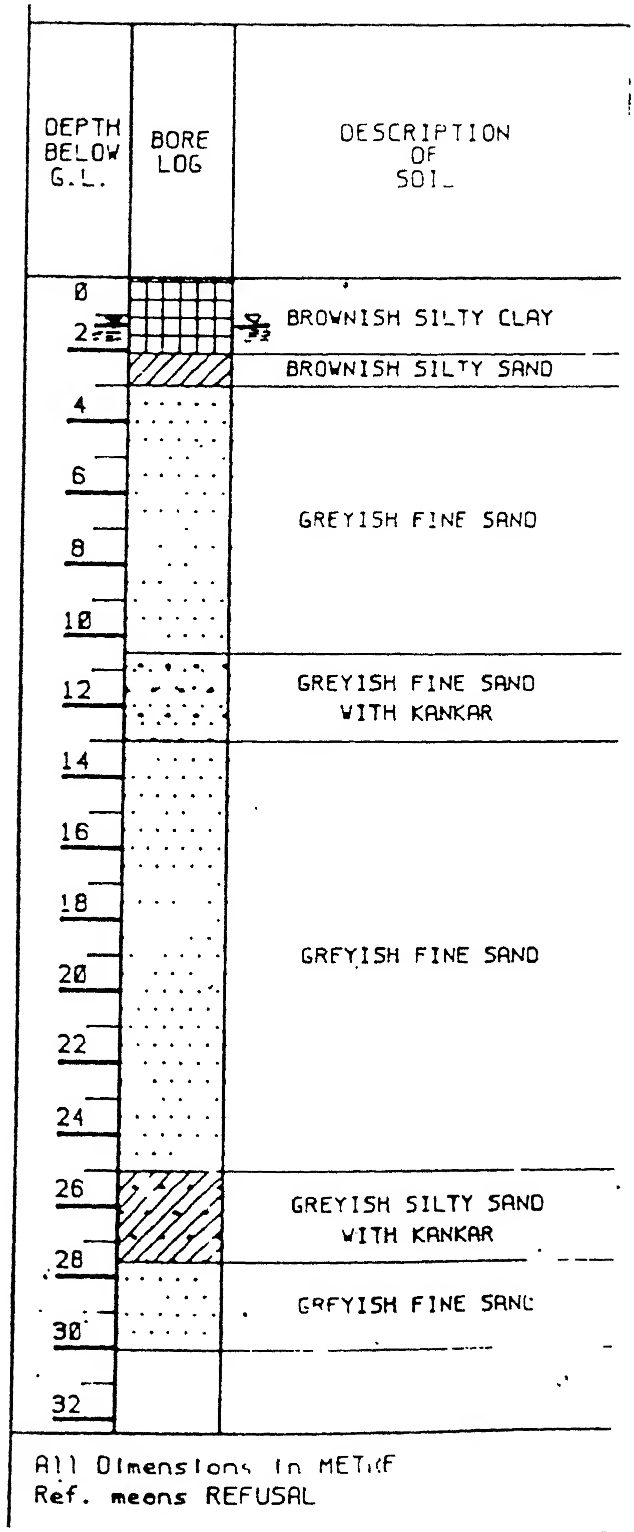

FIG - 3.9 BORELOGS: SHAHJAHNPUR

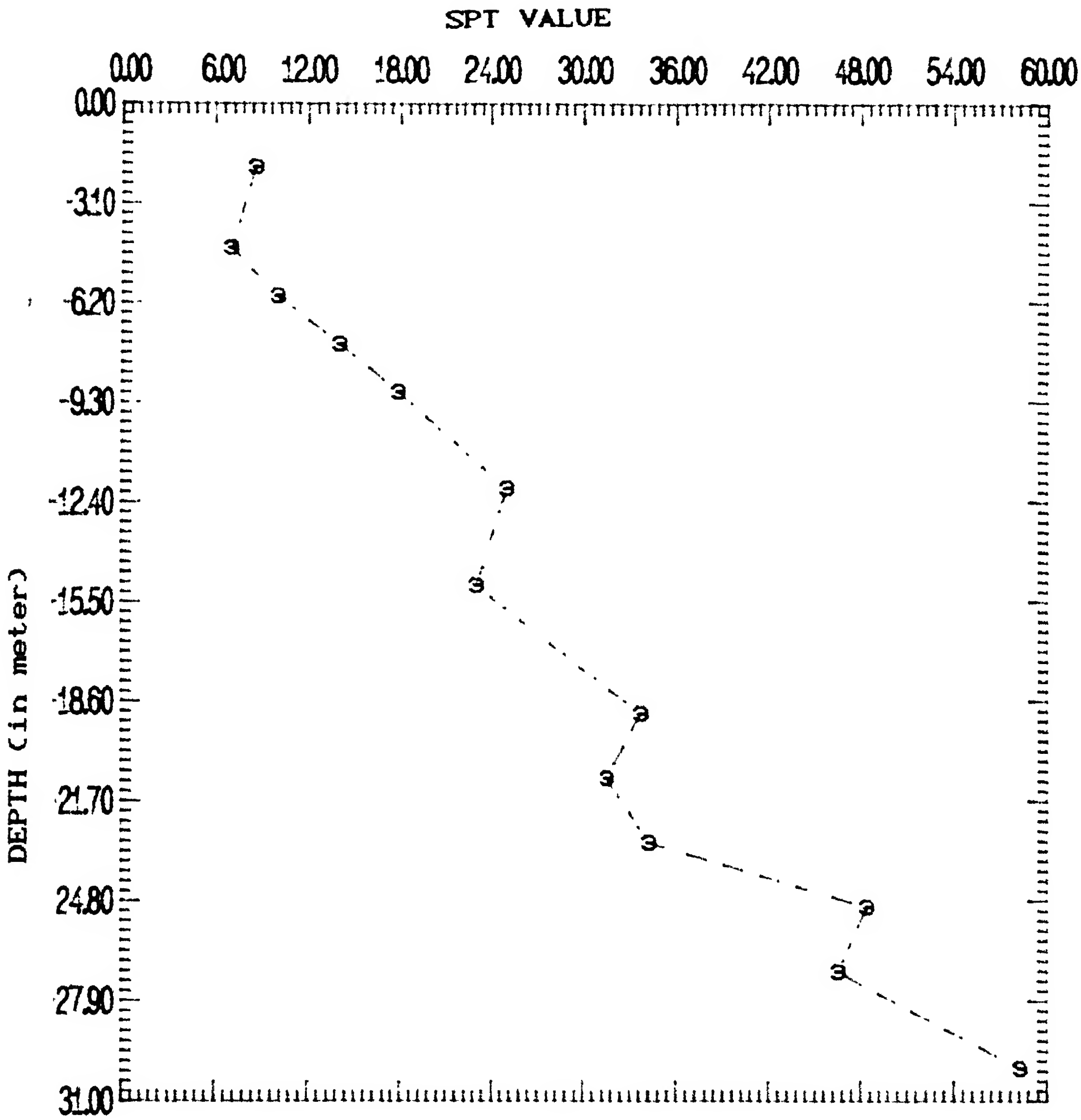

FIG - 3.10 **VARIATION OF SPT VALUE WITH DEPTH (SHAHJAHNPUR)**

ow plasticity(CL group) and silt clay of medium plasticity(CI roup). Thin lenses of non plastic silty sand and sandy silt(SM nd ML group) are also found at few depths. Kankar in varying roportion(nil to 47%) is also found. Clay content varies from 8 o 25%, whereas plasticity index varies from 6 to 16%.

The N-value in plastic strata ranges from 11 to refusal(N 50) indicating stiff to hard consistency of soil. In non plastic ayers N-value ranges from 22 to 50 indicating the deposit to be n medium to very dense state. The seasonal water table depth luctuates in between 4 to 5m.

As per IS Code 1893-1984 it lies in earthquake zone III. The ypical bore log in Raebareli is shown in Fig-3.11. From the arious bore log data collected, the variation of the average SPT alue with depth is shown in Fig-3.12.

.2.5 Kanpur This also forms the part of the Ganga basin lluvium plain. Top soil layer is silt or silty clay followed by ilty sand up to 8m. Beyond silty sand layer stiff silty clay ayer is encountered up to 30m. are found to be mixed with soil ayer at depths varying from 2 to 5m. The percentage of fine ontent is as high as ninety five. D_{50} varies from 0.014 to).068mm. The water table at various sites was located at depths arying from 1 m to 12m.

As per IS Code 1893-1984 it lies in earthquake zone III. The ypical bore log in Kanpur is shown in Fig-3.13. From the various

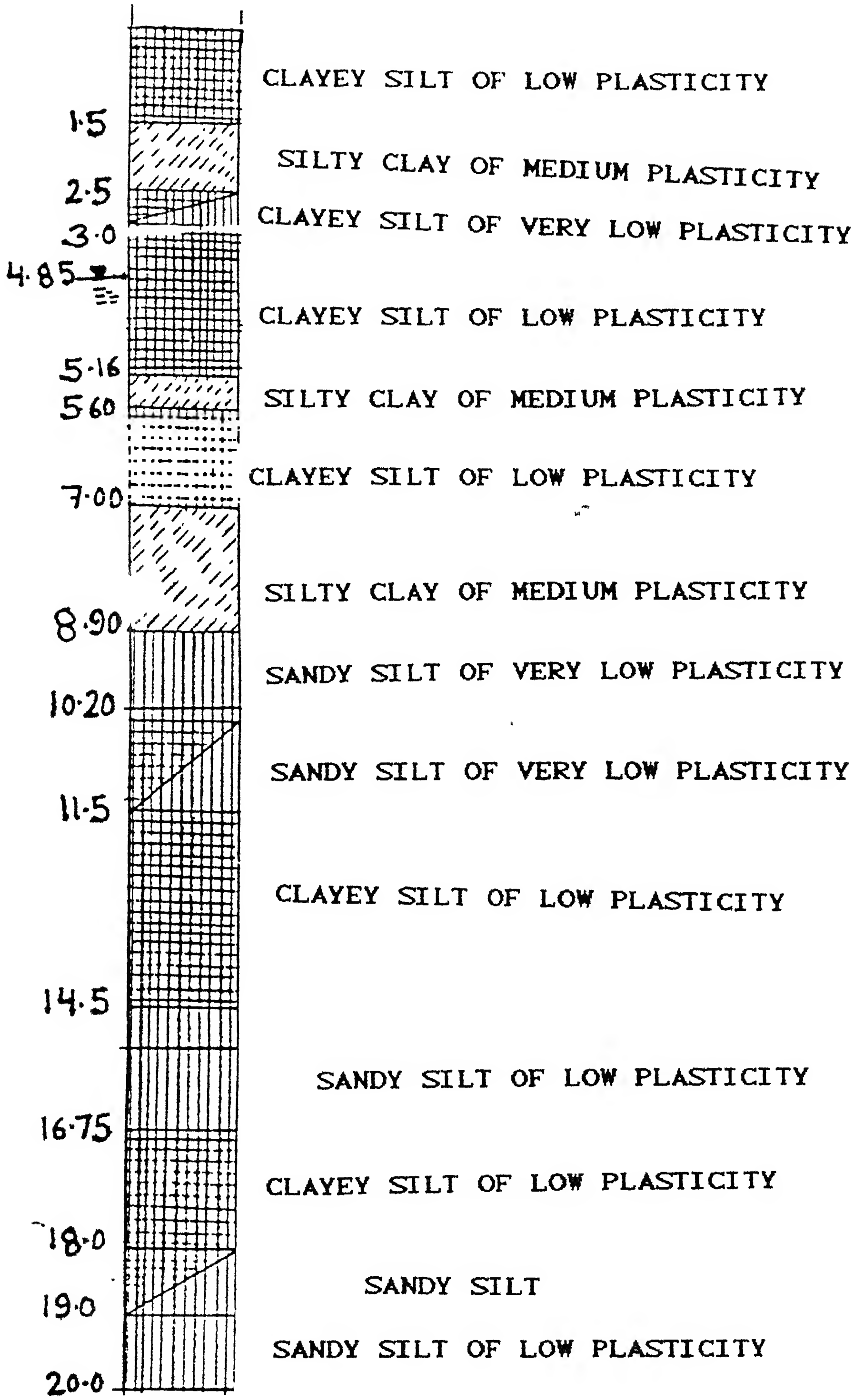

FIG - 3.11 BORELOGS: RAIBARELY

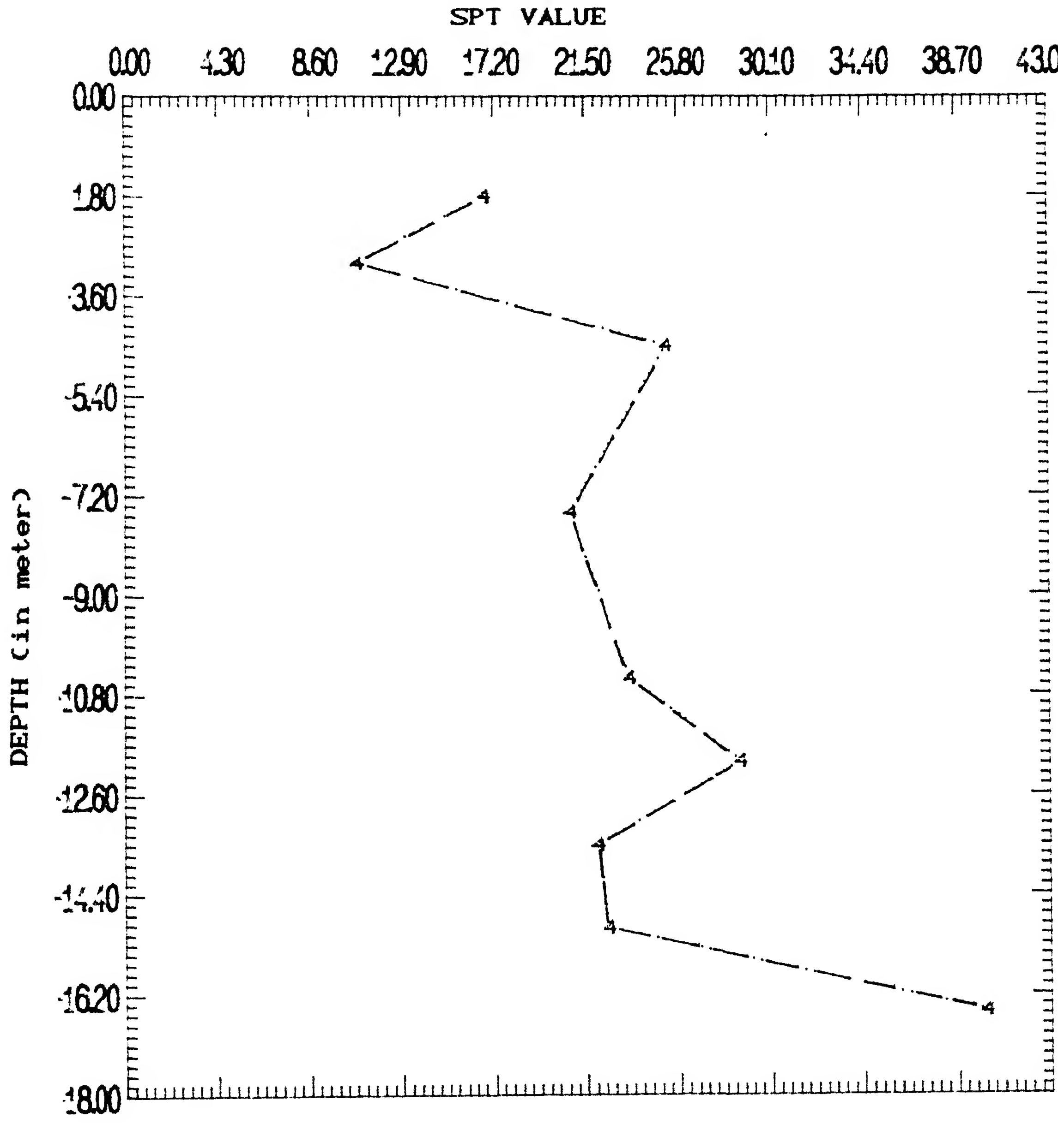

FIG - 3.12 **VARIATION OF SPT VALUE WITH DEPTH (RAIBARELY)**

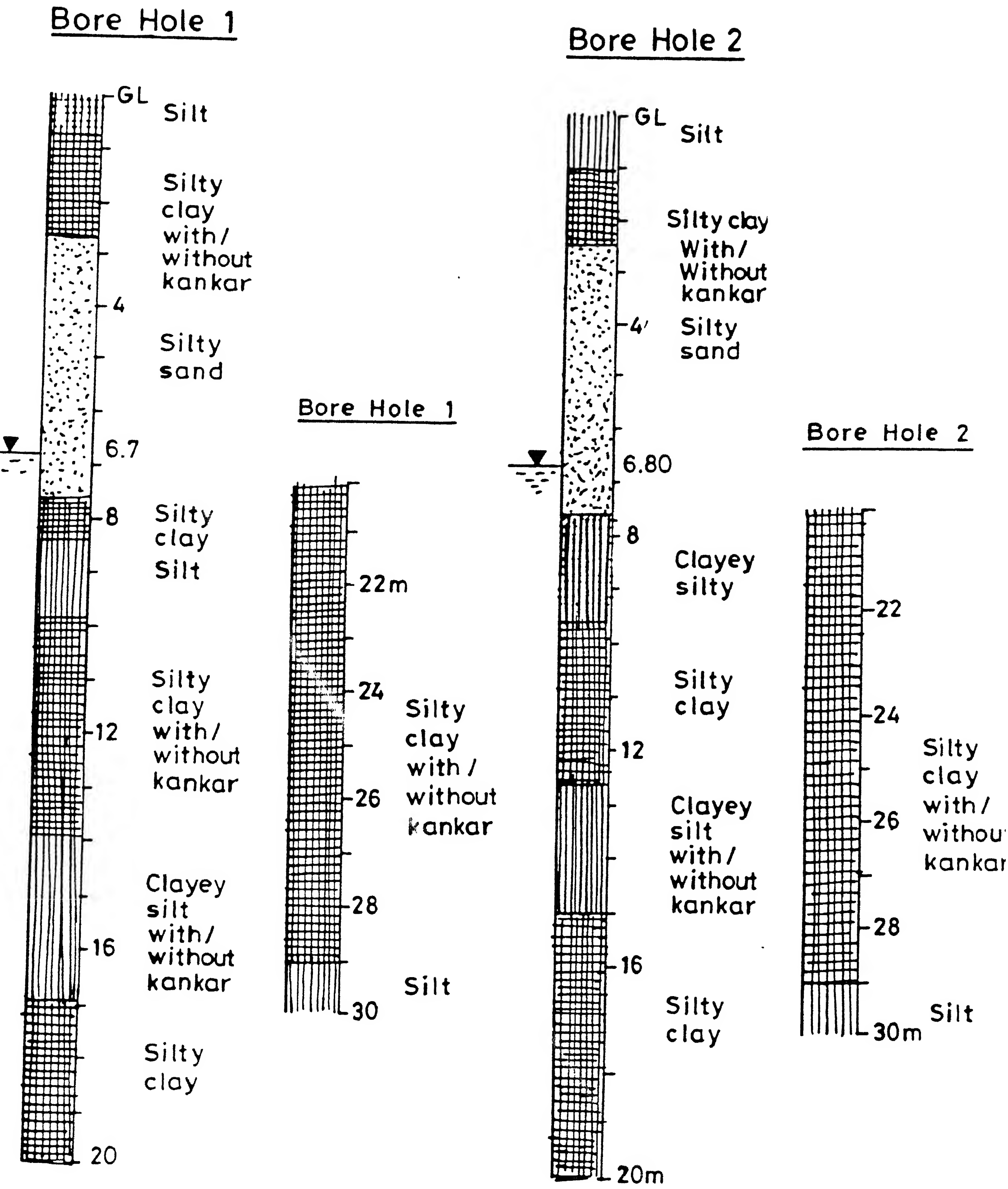

FIG - 3.13 BORELOGS: WIND TUNNEL PROJECT SITE, I.I.T. KANPUR

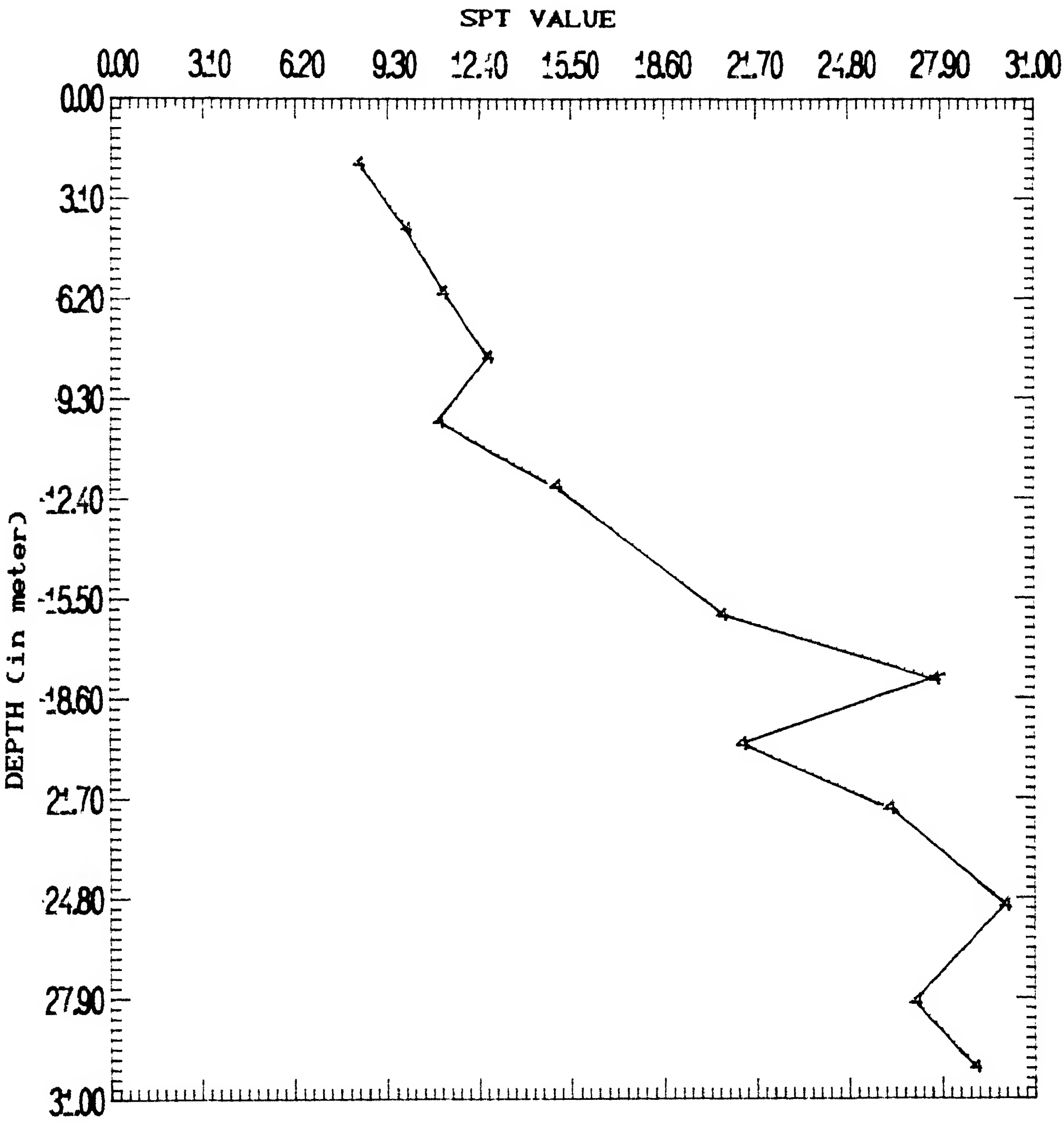

FIG - 3.14 **VARIATION OF SPT VALUE WITH DEPTH (KANPUR)**

bore log data collected, the variation of the average SPT value with depth is shown in Fig-3.14.

3.3 Evaluation of design horizontal earthquake acceleration

All the methods that have been discussed in chapter-2 require the likely earthquake magnitude and the ground acceleration for their application to predict the soil liquefaction potential. Our country as per IS Code 1893-1984 has been divided in different earthquake zones of different magnitudes and their epicentral position. It has also suggested the design coefficients which with importance factors gives the horizontal acceleration.

It should be clearly understood that the seismic coefficients recommended zone wise by IS Code 1893-1984 is of general nature and earthquake of high intensity may occur at a particular site, depending upon various factors. As such, IS Code 1893-1984 recommends that a rigorous analysis considering all the factors involve should be made to arrive at a suitable seismic coefficient.

Since earthquake acceleration could not be collected for all the chosen sites, choice of rigorous design ground acceleration analysis have not been carried out. As such semi-empirical relation proposed by (Seed et al., 1978) for alluvial deposit as given below is used to find horizontal ground acceleration. Soil deposits in all the sites that have been considered in this study are alluvial in nature, using of the suggested relationship is not likely to give rise to serious error.

$$\alpha_{max}(\text{gals}) = 32.1 \times 10^{0.254 \times M} \times \Delta^{-0.707} \qquad (3.1)$$

where

M – Richter magnitude

Δ – epicentral distance in Km

From the seismological map as shown in Fig 3.2 the likely earthquake magnitude and its epicentral distance from the concerned site have been worked out. The epicentral distance has been found by scaling directly from the map. For Shahjahnpur and Anola two epicenters and corresponding magnitudes are taken from (i) Etah fault (ii) Frontal Himalays. Similarly two epicenters and the corresponding earthquake magnitudes for Gorakhpur and Deoria are taken from Nepal Himalays. The computed values of the ground acceleration during earthquakes for the different chosen sites are presented in Table 3.2

α_{max} is divided by acceleration due to gravity to find α_{max}/g

Area	Earthquake magnitude	Epicentral distance (Km)	α_{max}/g
Gorakhpur and Deoria	7 8	150 180	0.045 0.07
Kanpur	6.5	120	0.04
Raebareli	6	15	0.15
Shahjahnpur and Anola	6 8	15 125	0.15 0.09

Table-3.2

The obtained ground acceleration values have been used subsequently for evaluating the liquefaction potential of different sites and the same is discussed in the next chapter.

CHAPTER 4

ASSESSMENT OF LIQUEFACTION POTENTIAL OF VARIOUS SITES OF UTTAR PRADESH AND COMPARATIVE STUDY OF DIFFERENT PREDICTIVE MODELS

4.1 General: In this chapter the liquefaction potential of various sites as described in chapter-3 has been evaluated by using different predictive models for the same as renunciated in chapter-2

A computer program written in fortran 77 has been developed for such predictions using all the methods as mentioned. The program has the ability to choose the appropriate methods based on the various available input parameters. If the input data is not suitable for a particular method the program would flash a message of its unsuitability. The various input parameters which are to be given in the program are, depth at which the liquefaction potential is to be found, depth of water table, unit weight of soil, clay content(%), plasticity index, fine content(%), D_{50}, type of soil(sandy or sandy loam), whether it is near field or far field, a_{max}/g, earthquake magnitude, number of stress cycles required to which causes liquefaction, SPT N-value.

An average strata profile has been worked out from tHe various bore-log data collected for a particular site. The same has been used for evaluating the average liquefaction potential of each site and then averaging the results for either the subdivision or the district as the case may be.

As for example in Gorakhpur Sadar bore log data for the sites namely Nichaul, Alinagar, Basaratpur,Charuchandrapuri etc are collected. Again for each site three bore-logs data are available. With the information collected from all these bore hole at a particular site, factor of safety versus depth against liquefaction is computed for each site. Finally these values are averaged for the entire Gorakhpur Sadar.

However, the program is capable of using separate values and evaluate liquefaction potential of individual site as per user requirement.

In case all the input parameters are available at all depths then the program is capable of finding the liquefaction potential based on the soil parameters at any particular depth. But if any data is missing then the values of the different soil parameters has been assumed to be same as that of its nearest location and used in the analysis.

As there is wide seasonal fluctuation of the depth of the water table the analysis has been carried out for the case where the water table is at at a higher level giving the worst condition of higher shear stresses during earthquake. Chang theory(1989) needs textural classification of the soil deposits as an input parameter. The same has been fed as input value based on triangular classification system (Taylor,1946).

Averaged N-value is corrected for overburden pressure as per

requirement of different methods. Variation of average N with depth is shown in the Figs-3.4, 3.6, 3.8, 3.10, 3.12 and 3.14. All input parameter units are converted to appropriate units as per the requirement of different methods.

Some of the assumptions that have been made in applying these methods are given below:

(i) Bolton method(1971) (a) stress ratio (Fig-2.2.2 to Fig-2.2.4)causing liquefaction are considered to be constant below particular value of D_{50} for which such curves are given. It is justified because Martin et al.(1978) found that stress ratio remains almost constant with D_{50}. (b) for earthquake magnitude less than 7.0 number of significant stress cycles considered are 10.0.

(ii) In Chang method(1989) for earthquake magnitude less than 7.0, $\overline{N}$ for the near field is 6.0 and for the far field is 8.0.

(iii) In Shibata and Teparaksa(1988) CPT q_c value are required; since for the chosen site no such insitu data are available , to use the method q_c versus depth values are found from SPT N-values using correlation valid for such deposits at different depths. This was done for Shahjahnpur and Anola. The correlation is established for such area are;

$$q_c = 2.5 \times N \qquad \text{(for depths < 12 mt)} \qquad (4.1.1)$$

$$q_c = 3.5 \times N \qquad \text{(for depths >12mt)} \qquad (4.1.2)$$

where N - SPT value

4.2 Procedure of using different methods:

Except Chang method(1989), in all other methods first of all total stress(σ_o) and effective stress(σ_o') is found out.The following steps are then followed in computing the liquefaction potential using the various methods:

Bolton and Idriss method(1971):

(1)For the desired depth, γ_d is found out from the Fig-2.2.1.

(2)With known value of α_{max}/g and calculated σ_o and σ_o', τ_{av} is found out from the equation (2.2.4).

(3) With the chosen value of earthquake magnitude number of cycles are taken from the table-2.2.1.

(4)Stress ratio is found from the appropriate Fig-2.2.2 to Fig 2.2.4 based on the D_{50} value.

(5)D_r is calculated from the equation(2.2.7)using the SPT value which is corrected for the overburden pressure.

(6)With above value of D_r, C_r is found out from the fig(2.2.5).

(7)With the above calculated value (stress ratio, C_r,D_r) stress ratio of the soil causing liquefaction is calculated from the equation(2.2.6).

(8)From the values calculated in steps(2 and 7) factor of safety is found out from the equation(2.2.8).

Ishihara Method(1979):

(1) γ_d is found out from the equation (2.3.2.a).

(2)With the known value of σ_o and σ_o' ,α_{max}/g, average shear stress developed during earthquake is found out from the equation(2.3.2)

(3)Cyclic strength is calculated from the equation(2.3.1) with known plasticity index, clay content, corrected N-value. N value

is corrected for overburden pressure from the equation(2.3.1.a)

(4)Factor of safety is found out from the equation(2.3.3).

The method is not applicable for fine content having zero value.

Tatsuoka et al.(1980):

(1)Using D_{50} value appropriate equation(2.4.1,2.4.2) is chosen and cyclic strength of the soil is found out.

(2)The cyclic stress ratio produced by the earthquake is calculated from the equation(2.4.6).

(3)Factor of safety is found out from the relationship(2.4.3) using the values calculated in the above steps.

Method is not valid for $D_{50} \leq 0.04$ mm.

Tokimatsu and Yoshimi(1983):

(1) At desired depth γ_d is found out from the relationship (2.5.1.b).

(2)γ_n is calculated from the equation(2.5.1.c) with known value of earthquake magnitude.

(3)Average cyclic stress ratio is found with the help of values calculated in above steps(1 and 2).

(4)SPT N-value is corrected for overburden pressure using equation(2.5.4).

(5)Cyclic strength of the soil deposit is calculated from the equation(2.5.14).

(6)Factor of safety is found out using equation(2.5.15).

Method is not valid for the fines content greater than 60.0%.

Shibata and Teparaksa method(1988):

(1)CPT q_c value is corrected for overburden pressure by using equation(2.6.1).

(2)With the known earthquake magnitude and acceleration, cyclic stress ratio is found from the equation(2.6.2).

(3)Using the above value calculated in step(2) critical CPT value $(q_c)_{cr}$ is found out from the equation(2.6.6). Care should be taken in using the above equation(2.6.6) for different D_{50} value.

(4)$(q_c)_{cr}$ is corrected for overburden pressure to find corrected $(q_c)_{cr}$ value from the equation(2.6.1).

(5)A graph is drawn between$(q_c)_{cr}$ and q_c versus depth. Zone where $(q_c)_{cr}$ exceeds q_c gives the zone of liquefaction.

(6)Factor of safety is found out from the equation (2.6.7).

Chang method(1989):

(1) To find the $\overline{N}$ corresponding to any earthquake magnitude that have not been tabulated, linear interpolation technique has been adopted.

(2)With the known SPT value and type of soil, appropriate equation(2.7.1-2.7.4) are chosen for finding SCPT(q_c) and Wave velocity(V_s).

(3)Whether a site under consideration with the known epicenter is in near field or in far field is determined.

(4)Critical SPT value is found out from the table(2.7.1) using earthquake magnitude.

(5)With the known parameter calculated above in step(4) critical wave velocity($\overline{V_s}$) and critical static point resistance($\overline{q_c}$) are calculated. The site is considered non-liquefiable if $\overline{V_s}$ and $\overline{q_c}$value are less than v_s and q_c.

4.3 Results and Discussions

The variation of factor of safety with depth against liquefaction as determined by using the different theories described in chapter 2 and section 4.3 of this chapter and the collected SPT data for the various chosen sites as reported in chapter 3 are presented site wise in Fig-3.4, Fig-3.6, Fig-3.8, Fig-3.10, Fig-3.12, Fig-3.14. In each of the presented figures a vertical line corresponding to the factor of safety of 1.5 is drawn; for any particular depth, factor of safety values lying to the right of the line signify that soil at that depth has adequate safety against liquefaction. In these figures the following nomenclatures are used,

BOLTON: Bolton and Idriss(1971)

ISRA: Ishihara(1979)

TITY: Tatsuoka et al.(1980)

TOKI: Tokimatsu and Yoshimi(1983)

SHITE: Shibata and Teparaksa(1988)

E.D: Epicentral distance in km

M: Earthquake magnitude

Safe Line corresponding to a factor of safety equal to 1.5.

Gorakhpur Sadar: The fig-4.3.1 shows the variation of factor of safety with depth for earthquake magnitude 7 corresponding to epicentral distance of 150 km. Similar variation for M = 8 and E.D = 180 km is presented in Fig-4.3.2. Fig-4.3.1 shows that according to all the theories the soil will not liquefy at all through out the deposit. The TITY method predicts the highest factor of safety

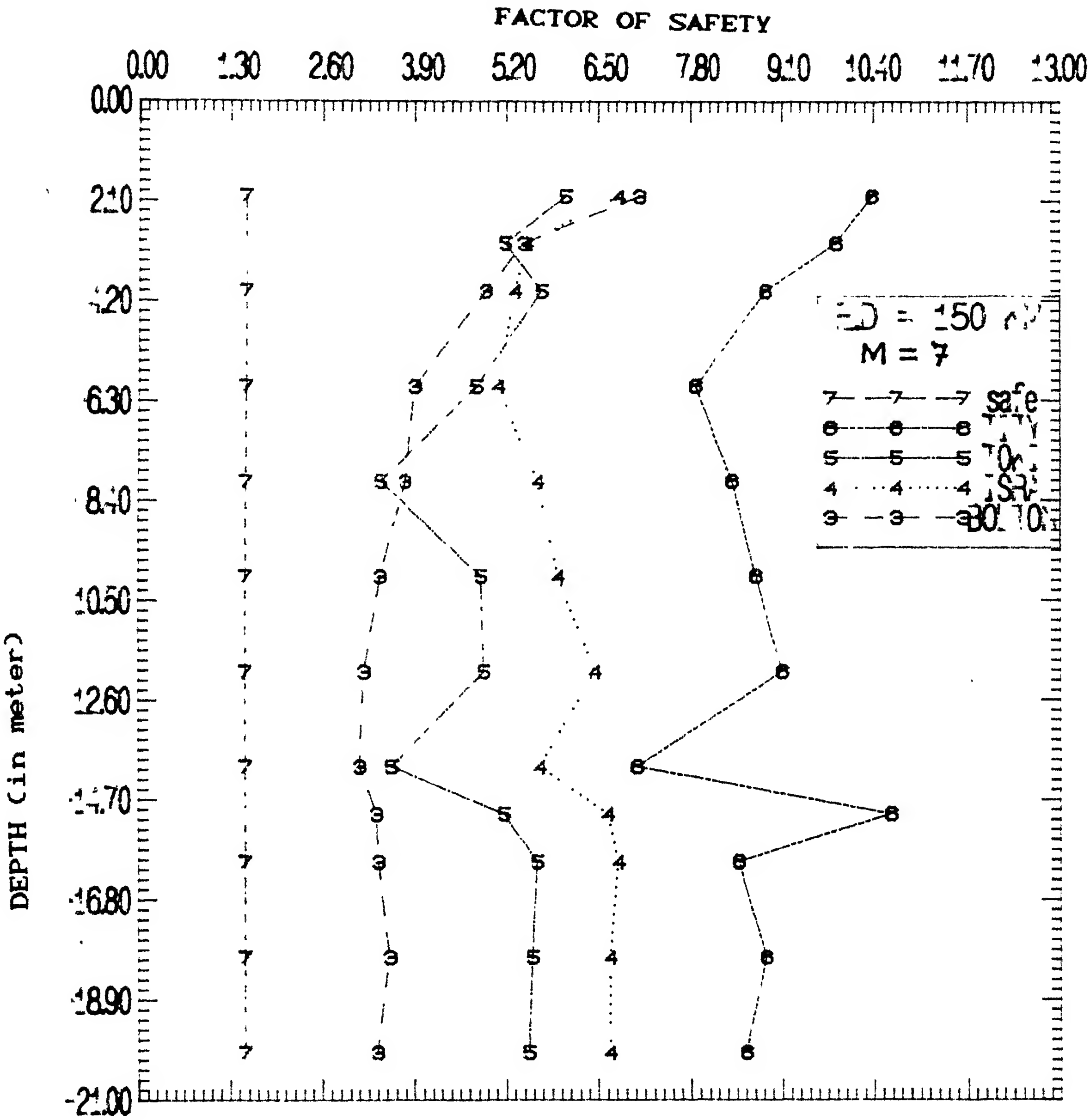

FIG-4.3.1 RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (GORAKHPUR)

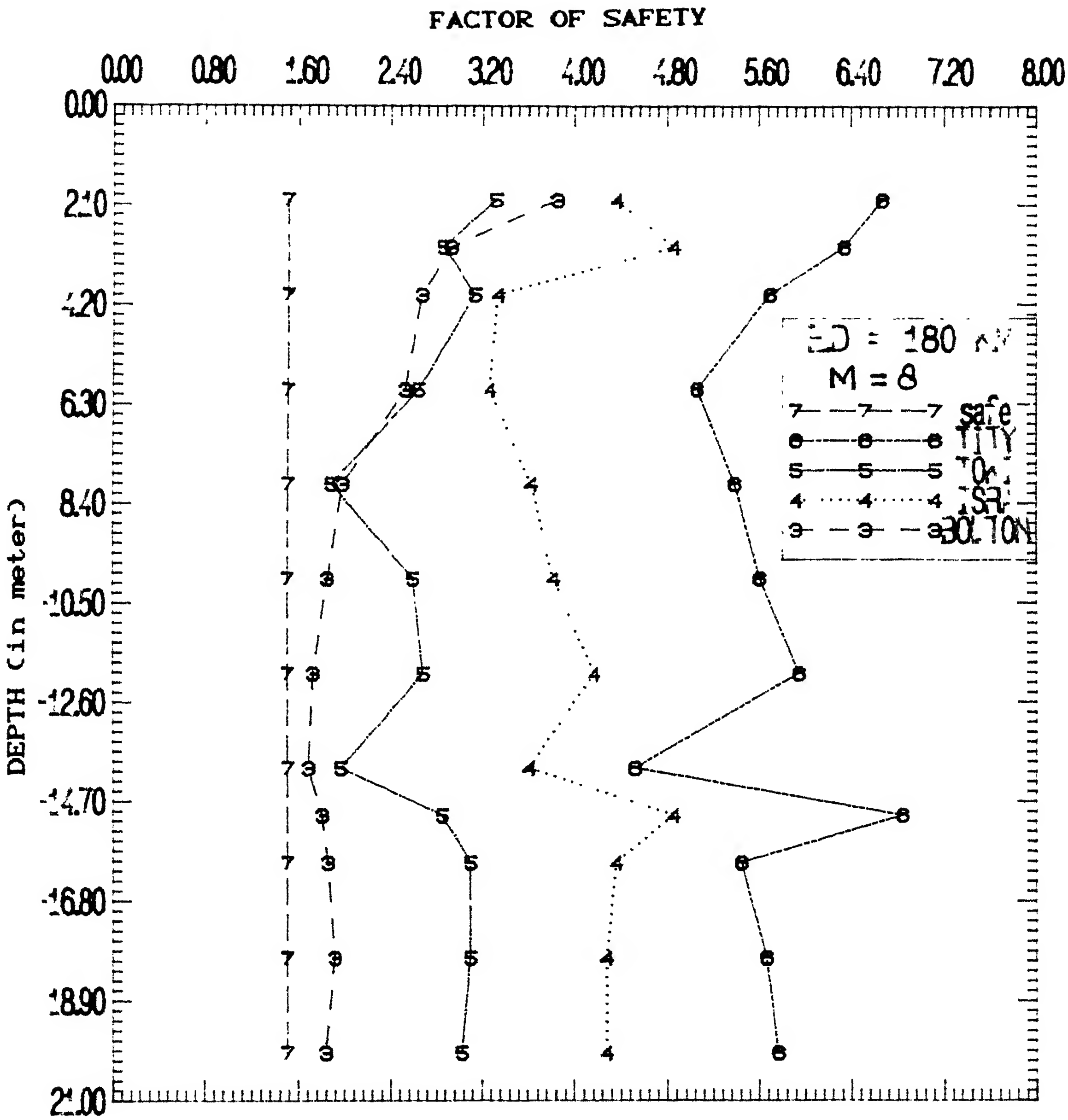

FIG-4.3.2 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (GORAKHPUR)**

where as BOLTON method in general predict the lowest value over almost the entire depth. Except the TITY method all the other three methods predict factor of safety values which do not vary much up to about 3.0m. Fig-4.3.2 shows similar behaviour. However in this case the predicted values by TOKI and BOLTON are in close agreement over a depth of about 8.0m. TITY and ISRA methods predict similar variation of factor of safety with depth but differ in magnitude.

From the above study it can be concluded that the Gorakhpur soil is not prone to liquefaction when the ground acceleration is of the order of 0.07g(table-3.1) as per TITY, TOKI, ISRA and BOLTON methods.

But it should be noted that the chang theory(1989) predicts that Gorakhpur soil would liquefy based on different magnitudes.For M = 8 and E.D = 150 km the soil would liquefy from a depth of 2m to 20m whereas for M = 7 and E.D = 150km the value is 2.0 to 10m. As this method gives only qualitative information whether a soil would liquefy or not, factor of safety variation with depth for this theory could not be drawn.

From the above discussion it may further be inferred that Chang's method is the most conservative out of all the methods when applied to such type of deposits.

Deoria sadar: Fig-4.3.3 and Fig-4.3.4 indicate that soil is not likely to liquefy and has adequate safety against liquefaction. It is also seen from the figures that TITY method predicts the highest factor of safety whereas BOLTON method is the most conservative except at shallower depths. In Fig-4.3.3 no factor of safety values could be evaluated by TOKI method beyond 12m depth as the method is not applicable for the type of soils encountered in depths greater than 12m. For depths lower than 4m the TOKI method predicts factor of safety values much lower than those predicted by BOLTON method. Fig-4.3.4 shows similar trend in the factor of safety variation with depth. For this case(M = 8, E.D = 180 km) a ground acceleration value of 0.07g will be experienced and at a depth of about 18m factor of safety would just be adequate according to BOLTON method. According to Chang theory for both the earthquake magnitudes soil would liquefy up to a depth of 10m.

ANOLA: Fig-4.3.5 shows the factor of safety variation with depth for an earthquake of magnitude 8 with an epicentral distance of 125 km whereas Fig-4.3.6 shows the same for earthquake magnitude of 6 and epicentral distance 15 km. Both these figures show that for this site SHITE method is the most conservative; according to this method the soil would liquefy up to a depth of about 12m for M =8 and E.D = 125 km and up to a depth of 14m for M = 6 and E.D = 15 km. It is also seen that soil do not have adequate safety against liquefaction up to a depth ranging from 14m to 15.5m. This can be clearly demonstrated by the Fig-4.3.7 and Fig-4.3.8, where the cone resistance(q_{c1}) versus depth and

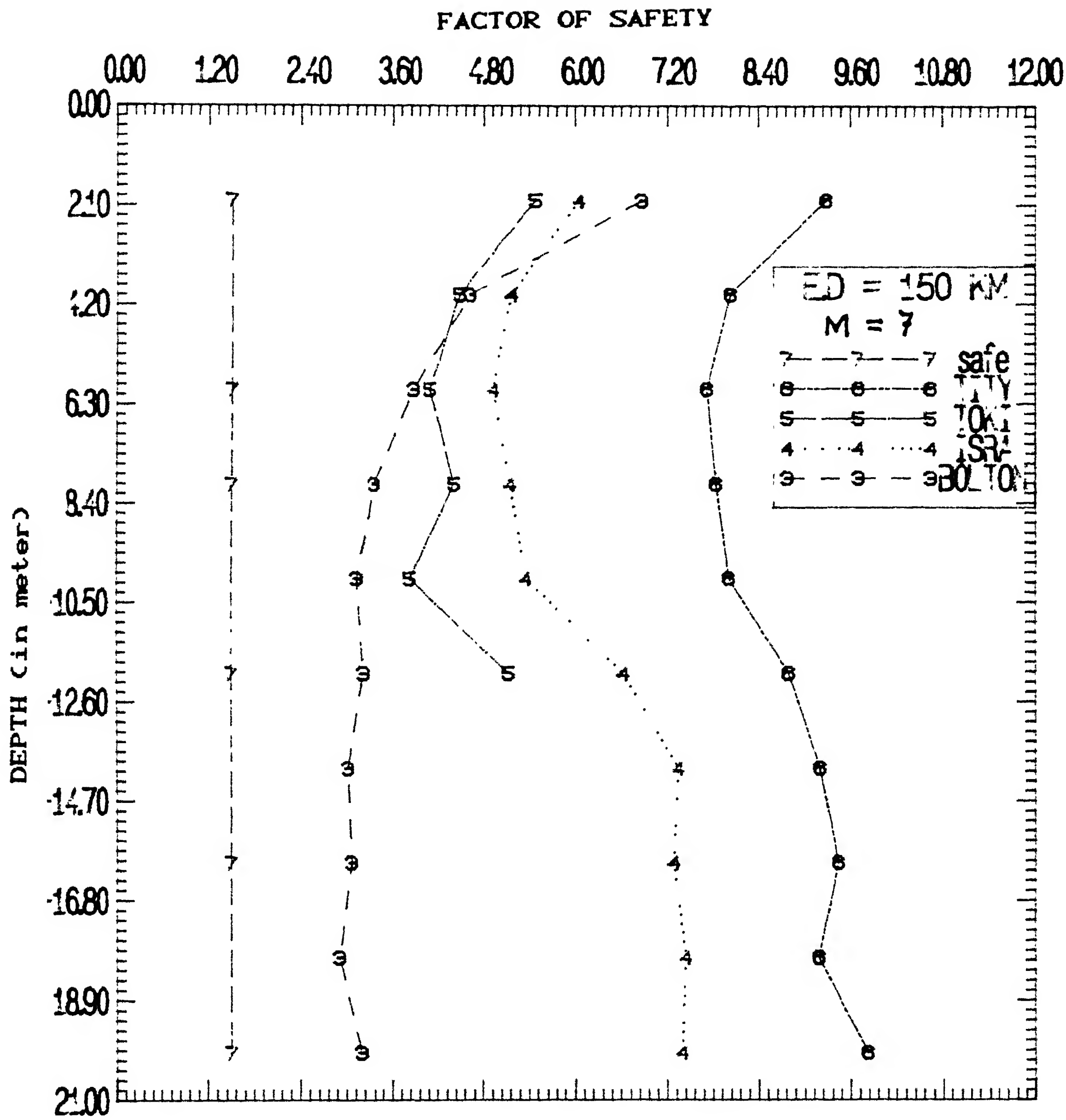

FIG-4.3.3 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (DEORIA)**

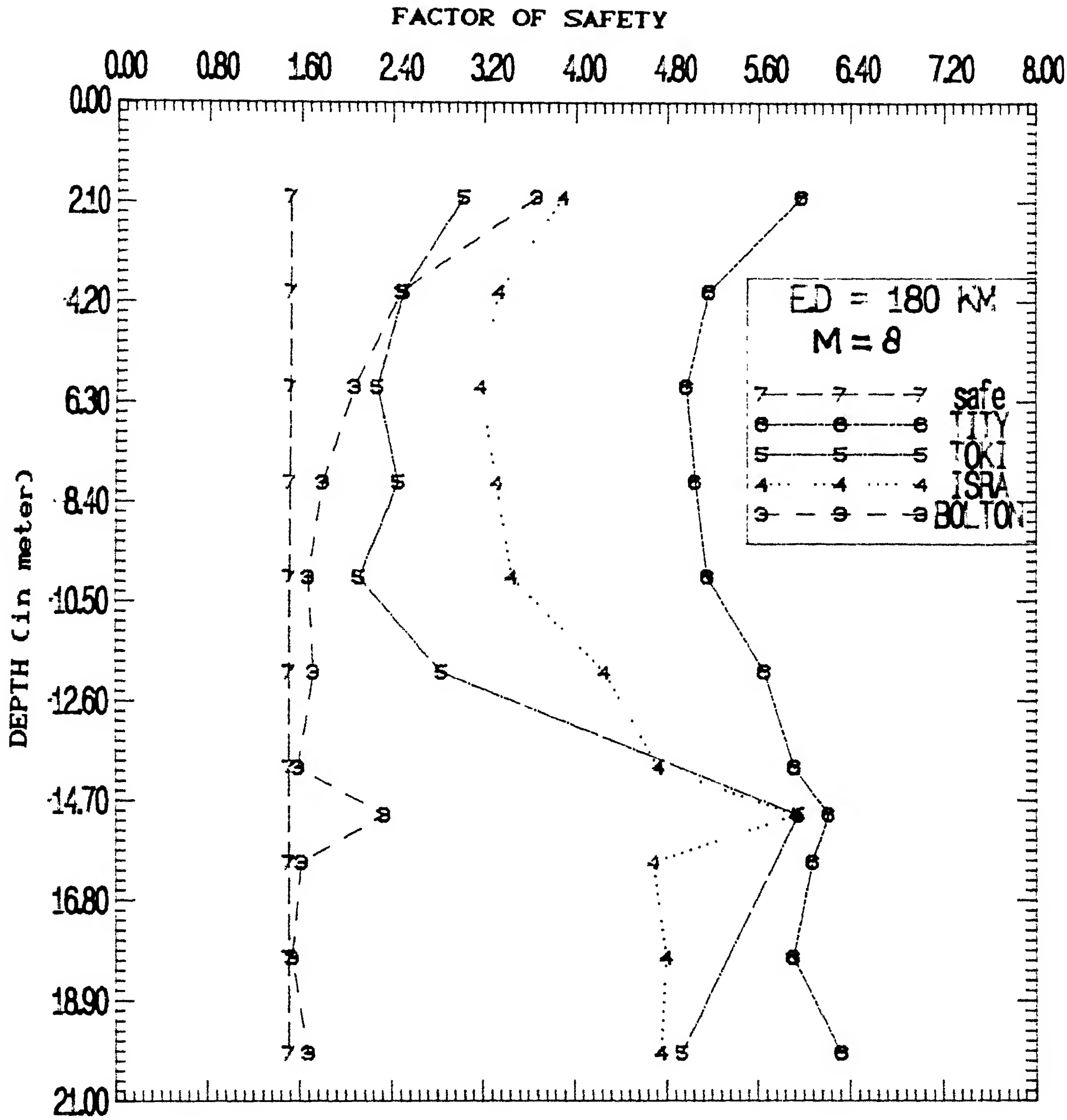

FIG-4.34 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (DEORIA)**

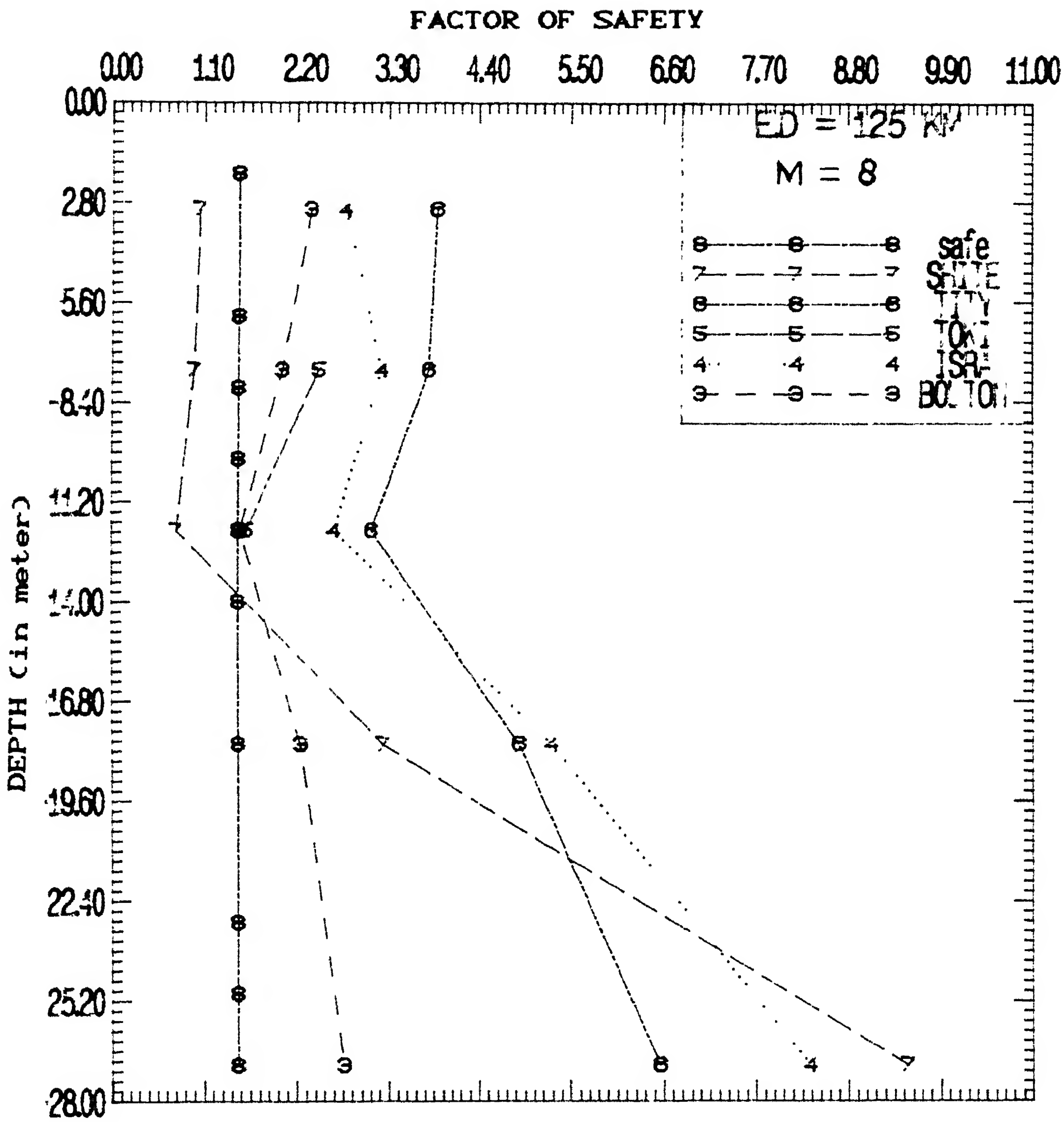

FIG-4.3.5 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (ANOLA)**

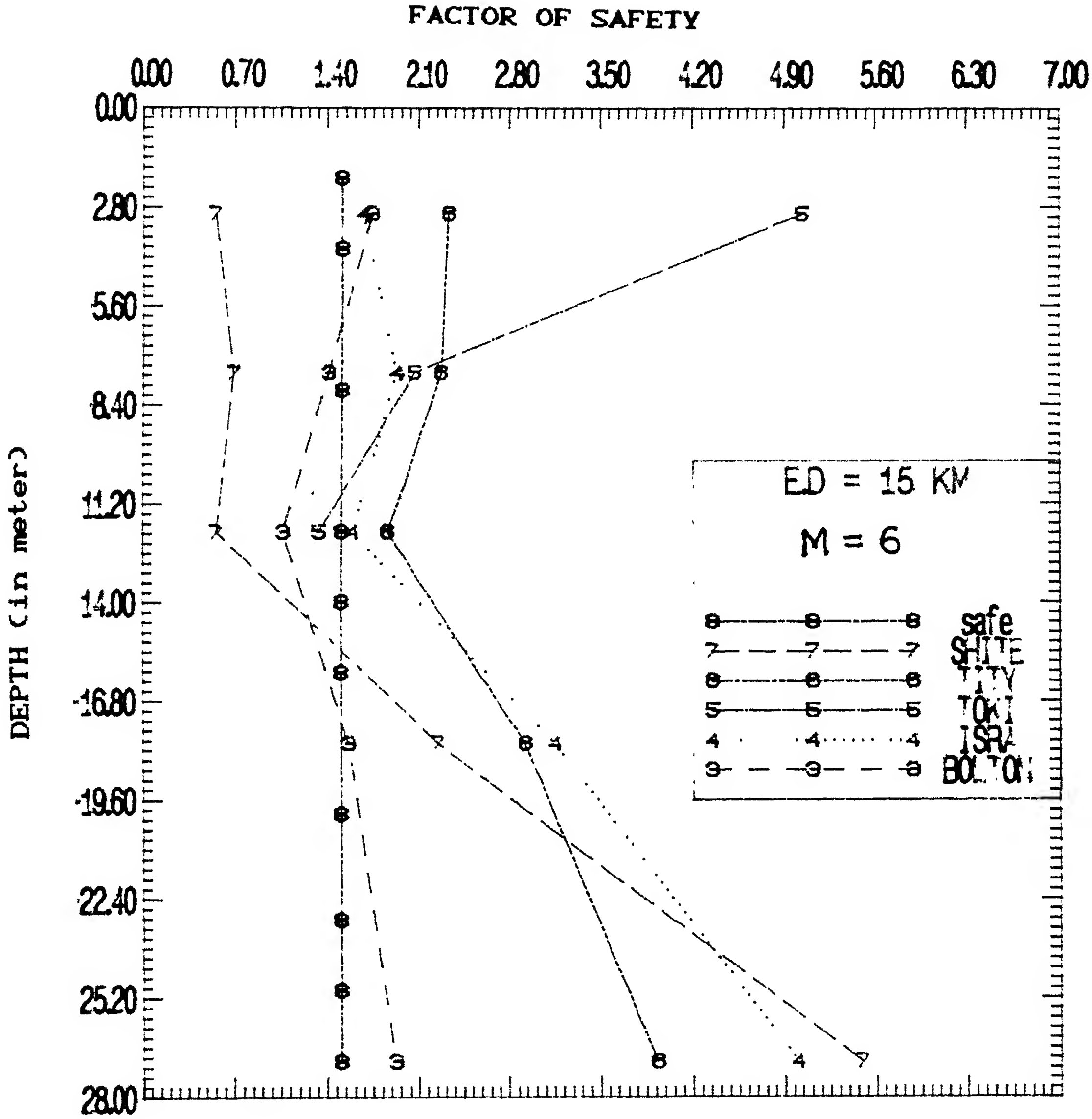

FIG-4.3.6 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (ANOLA)**

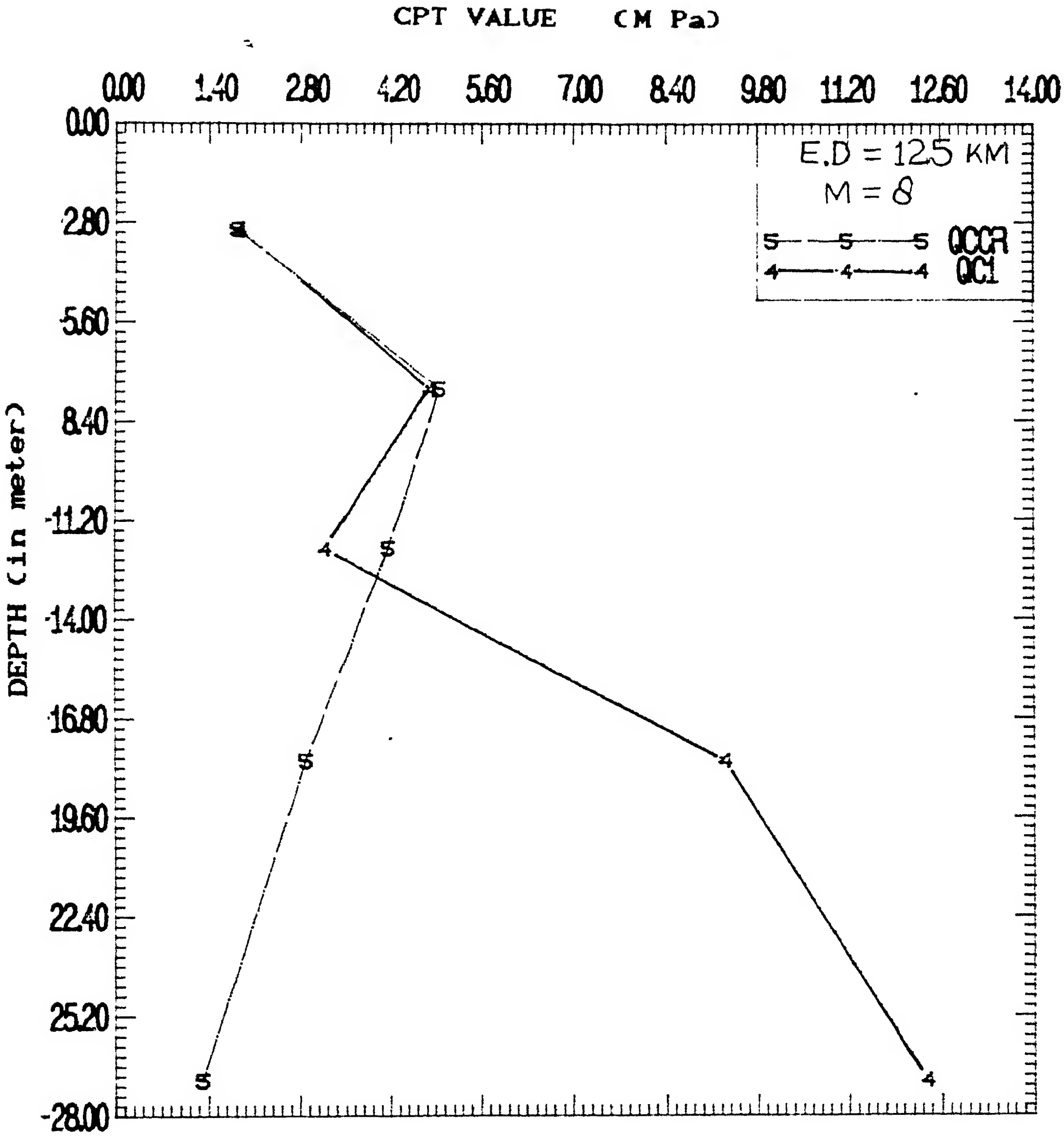

FIG-4.3.7 **LIQUEFACTION ZONE (SHIBATA ET AL. METHOD) (ANOLA)**

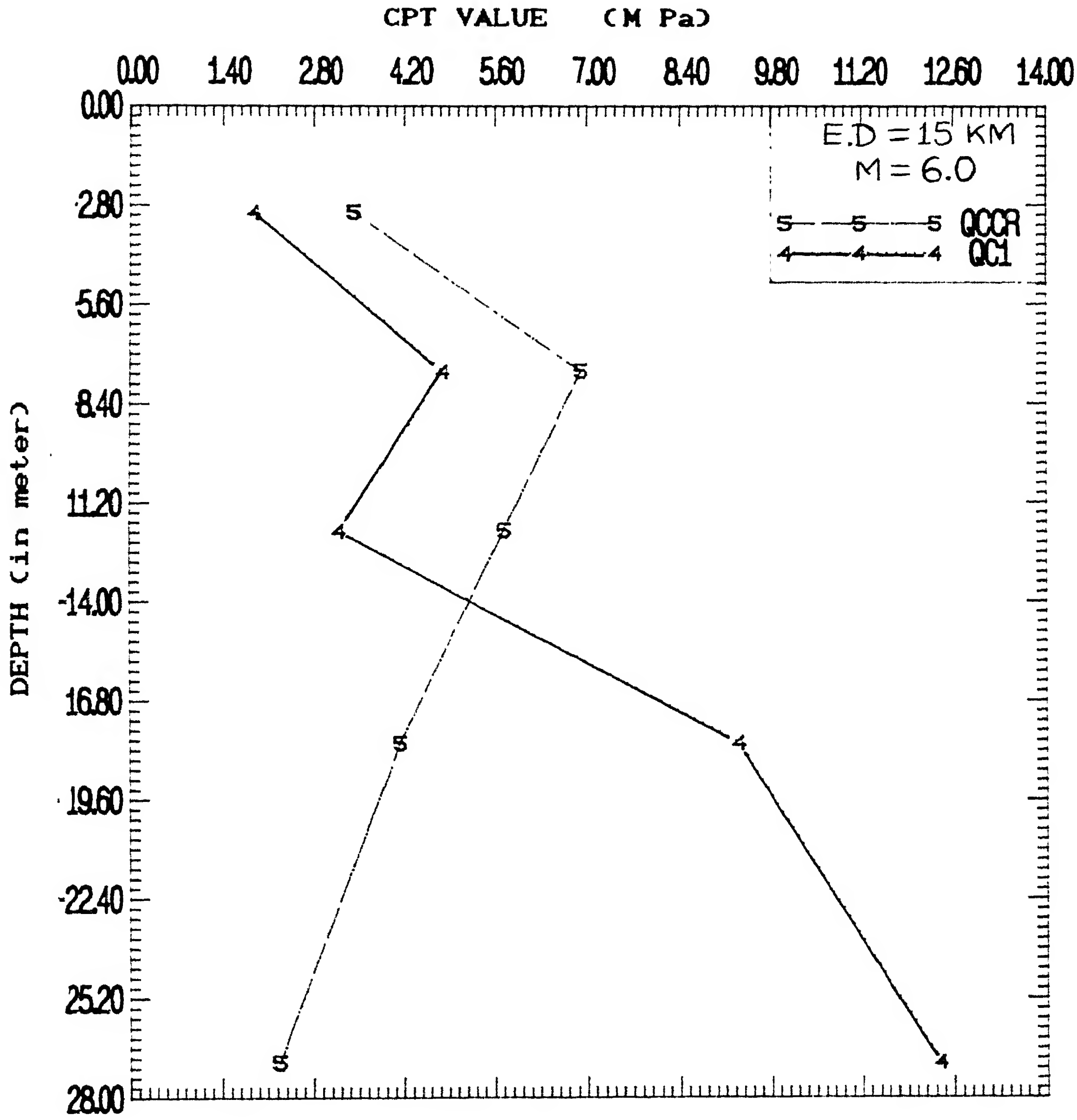

FIG-4.3.8 **LIQUEFACTION ZONE (SHIBATA ET AL. METHOD) (ANOLA)**

critical cone resistance($(q_c)_{cr}$) versus depth variation are shown. When cone resistance value is less than the critical cone resistance soil is likely to liquefy. From Fig-4.3.7 it can be seen that soil would liquefy up to a depth of 13m for M = 8, E.D = 125 km, the corresponding value for M =6, E.D = 15 km is 14.25m.

Below this range of depth BOLTON gives the most conservative estimate of factor of safety. From Fig-4.3.5 it can be seen that BOLTON method predicts that at about 12m depth the soil is having a factor of safety which is just sufficient. Marginal increase in the ground acceleration value due to the occurrence of either a higher magnitude earthquake or earthquake of same or lower magnitude at a nearby place may trigger a soil liquefaction. This is indeed the case as evident from fig-4.3.6 which shows that for M = 6 and E.D = 15 km, BOLTON method predicts that soil would liquefy and the liquefied zone is likely to extend from 6.5m to 17.0m. The figures show that TOKI method is applicable only over certain depth. Fog-4.3.6 shows that the soil does not have adequate safety factor below a depth of 11m. TITY and ISRA methods predict that the soil has adequate safety factor over the entire depth. According to Chang's method for M =6, E.D = 15km soil will liquefy up to a depth of 12m and for M = 8 ,E.D = 125km the soil deposit is not likely to liquefy.

Shahjahnpur: Fig-4.3.9 and Fig-4.3.10 show the factor of safety variation with depth corresponding to M = 6, E.D = 15km and M = 8,E.D = 125km respectively.

Fig-4.3.9 indicates that SHITE method provides the most

conservative analysis. According to this theory the soil deposit do not have adequate factor of safety up to a depth of about 21m; it can also be observed from this figure that the soil deposit will liquefy up to a depth of about 16m for M = 6,E.D = 15km. Like the site at Anola the conclusion drawn is further clarified through Fig-4.3.11 and Fig-4.3.12. It can be seen that the liquefied zone would extend up to a depth of about 12m for M = 8, E.D = 125km; the corresponding value for M = 6, E.D = 15km is about 15m.

Bolton method indicates that the zone of inadequate factor of safety would extend from about 6.5m to 18m depth. However, even though the deposit does not have adequate safety against liquefaction, the safety factor is never below the limiting value of unity below which soil liquefaction occurs. ISRA method shows that the zone of inadequate factor of safety extends from only 3.5m to about 6.5m. TITY method indicates the zone to be from about 3.0m to about 9.0m. TOKI method is not applicable up to depth of 4.5m; from 4.5m to 7.5m depth factor of safety is just adequate. Beyond 7.5m TOKI method predicts very high value of factor of safety in comparison to other methods. Use of Chang method indicates the deposit will not liquefy for M = 6, E.D = 15km whereas for M =8, E.D = 125km the soil is likely to liquefy at the zones extending from 4.5m to 12m and again from 19m to 23m.

Fig-4.3.10 shows that only SHITE method predicts inadequate safety factor extending up to a depth of about 16m for M = 8, E.D = 125km. All other methods(TITY, ISRA, TOKI, BOLTON) predict

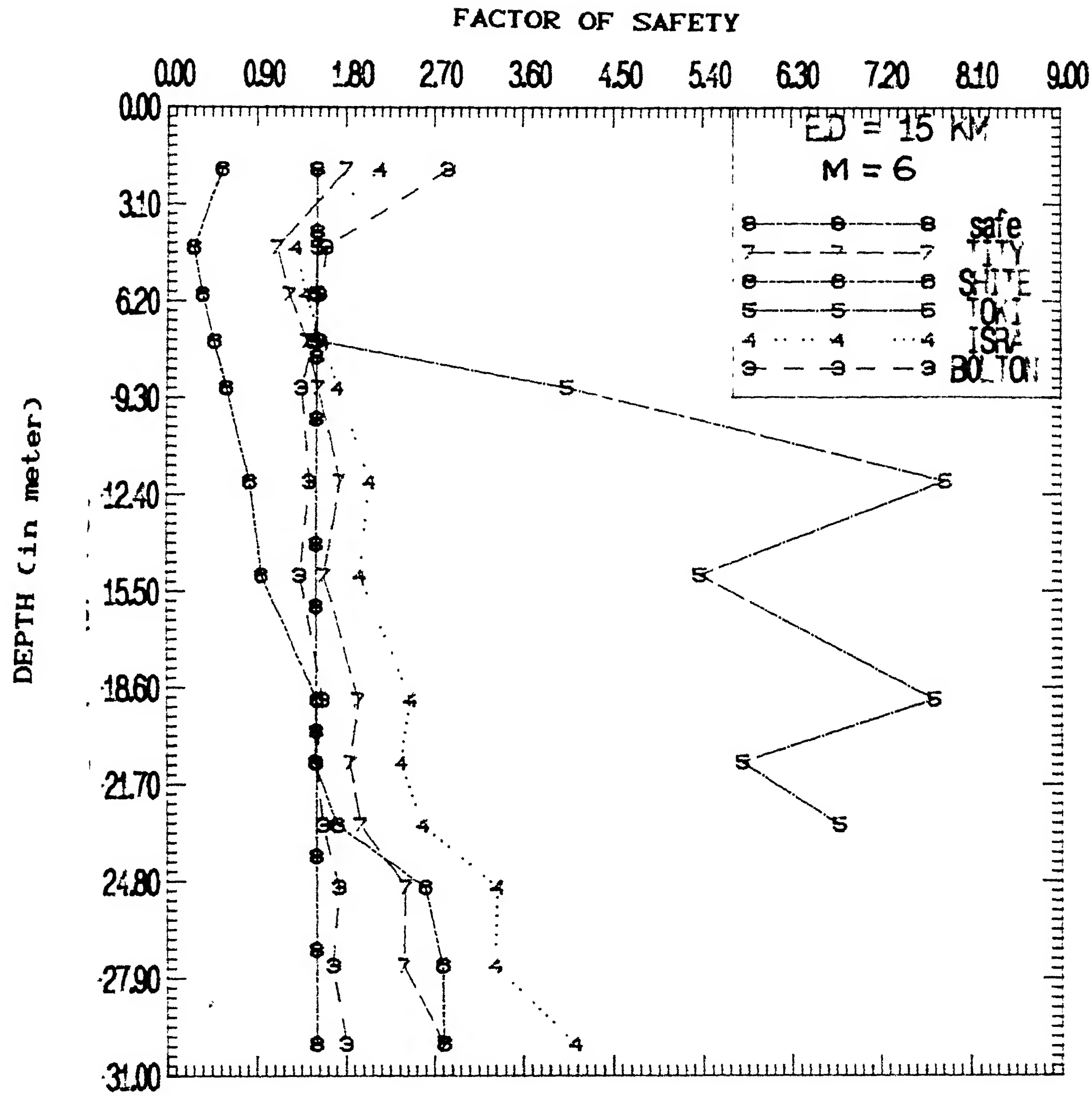

FIG-4.3.9 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (SHAHJHNPUR)**

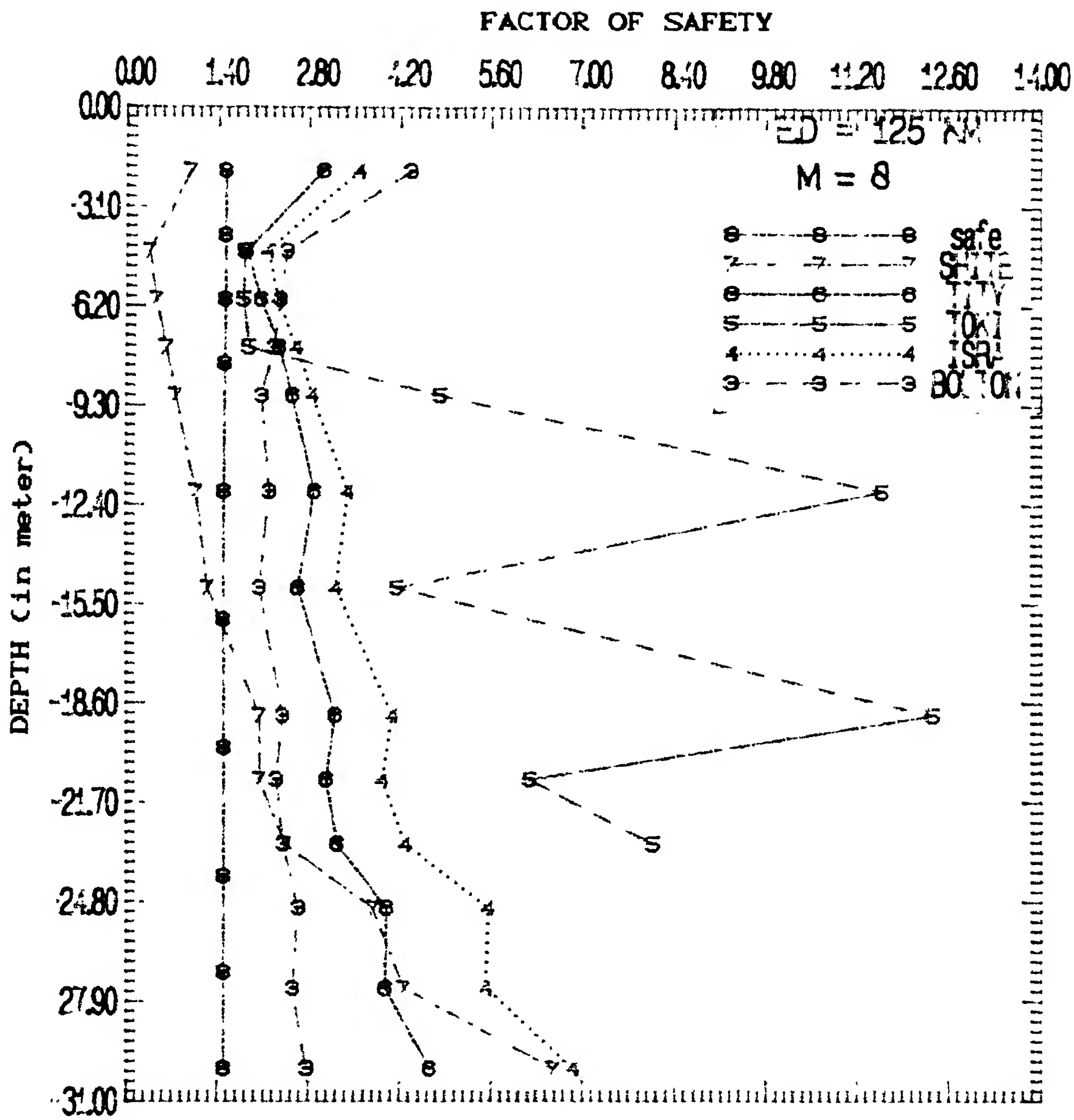

FIG-4.3.10 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (SHAHJHNPUR)**

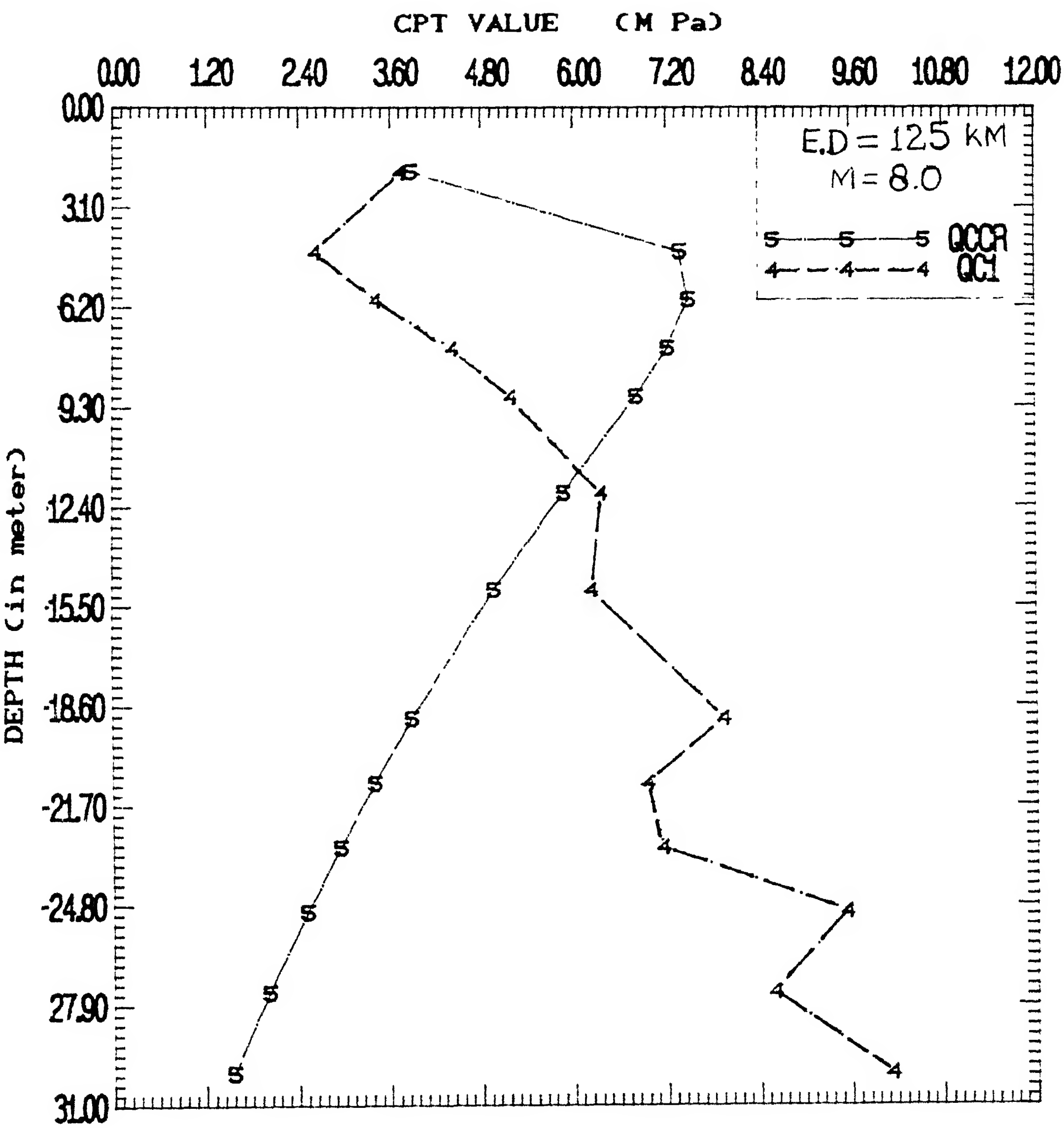

FIG-4.3.11 **LIQUEFACTION ZONE (SHIBATA ET AL. METHOD) (SHAHJAHNPUR)**

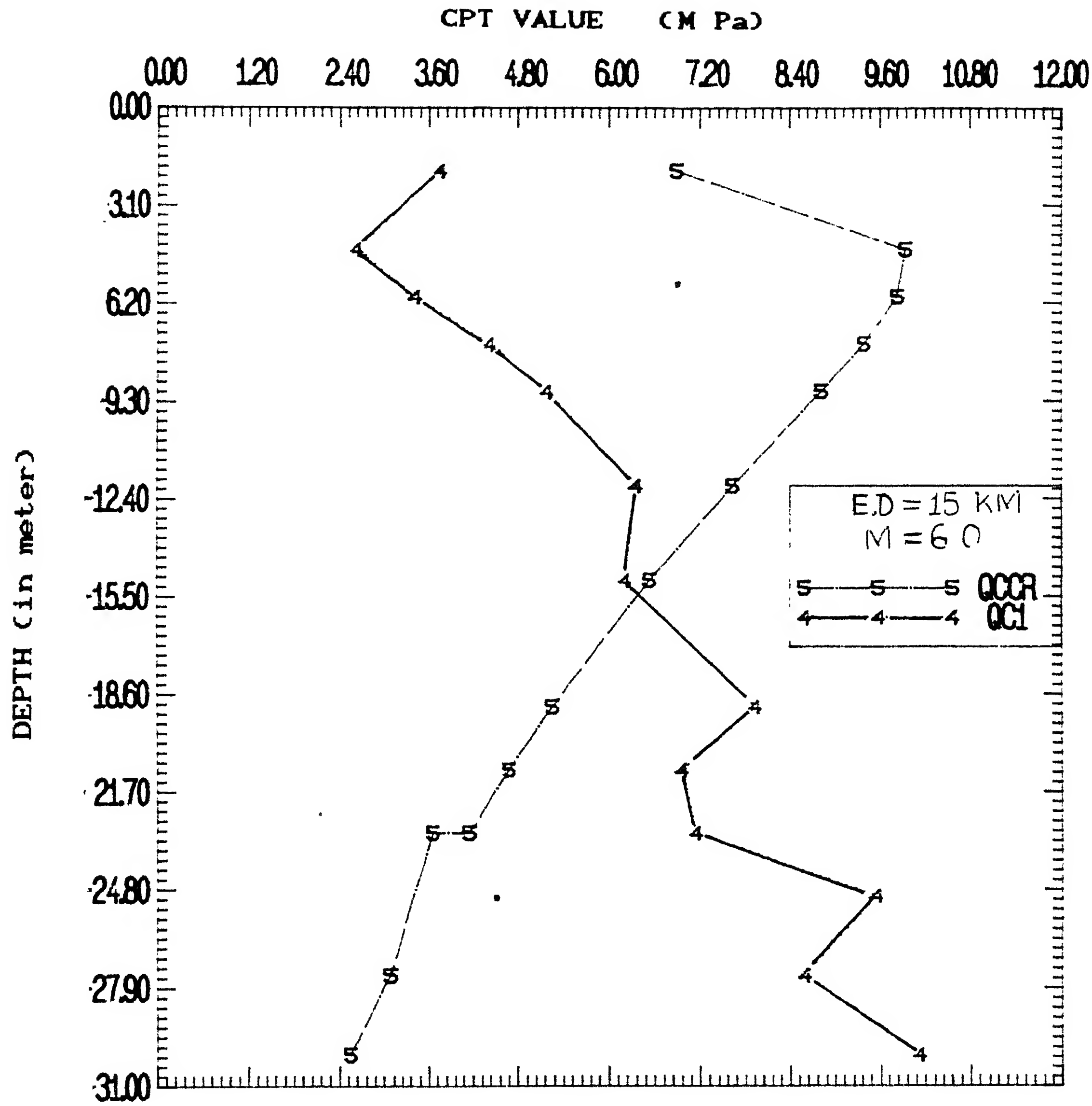

FIG-4.3.12 **LIQUEFACTION ZONE (SHIBATA ET AL. METHOD) (SHAHJAHNPUR)**

factor of safety values greater than 1.5 for the entire depth.However, like the previous case of M = 6, E.D = 15 km in this case also TOKI predicts substantially higher values in comparison to the values predicted by the other methods.

Raibarely: In this case analysis of data have been carried out using only TITY, ISRA and BOLTON method. TOKI method is not applicable in this deposits as the fine content is more than 60%. SHITE method could not be applied as no data was available.

The factor of safety variation with depth is shown in Fig-4.3.13. From the figure it can be seen that the obtained trend of variation is similar for both TITY and ISRA methods but the magnitude of the predicted factor of safety values differ substantially; both the methods predict that the soil deposit has more than adequate factor of safety over the entire depth. However, BOLTON method indicates that the soil does not possess adequate safety factor for depths greater than 5.5m. The figure indicates that the soil deposit ia almost at the limiting state over a zone extending from 10 to 14m and as such any marginal increase in the ground acceleration value during earthquake would trigger liquefaction. Chang method predict that soil will not liquefy at all.

Kanpur: From the Fig-4.3.14, for Kanpur soil deposits only TITY, ISRA and BOLTON method could be applied to find the liquefaction potential. Over the entire depths of the deposit under consideration TOKI method could be applied only over a

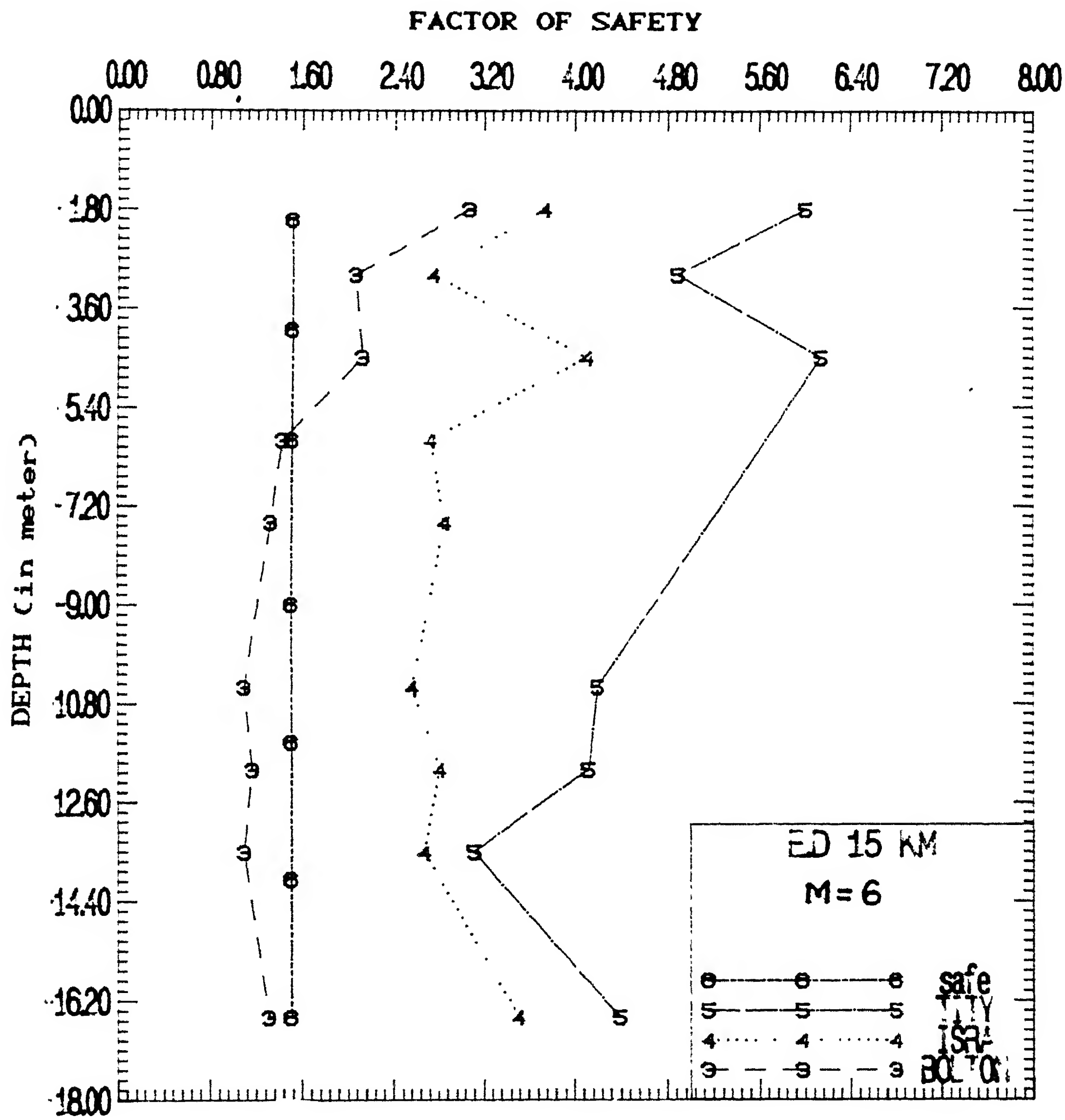

FIG-4.3.13 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (RAIBARELY)**

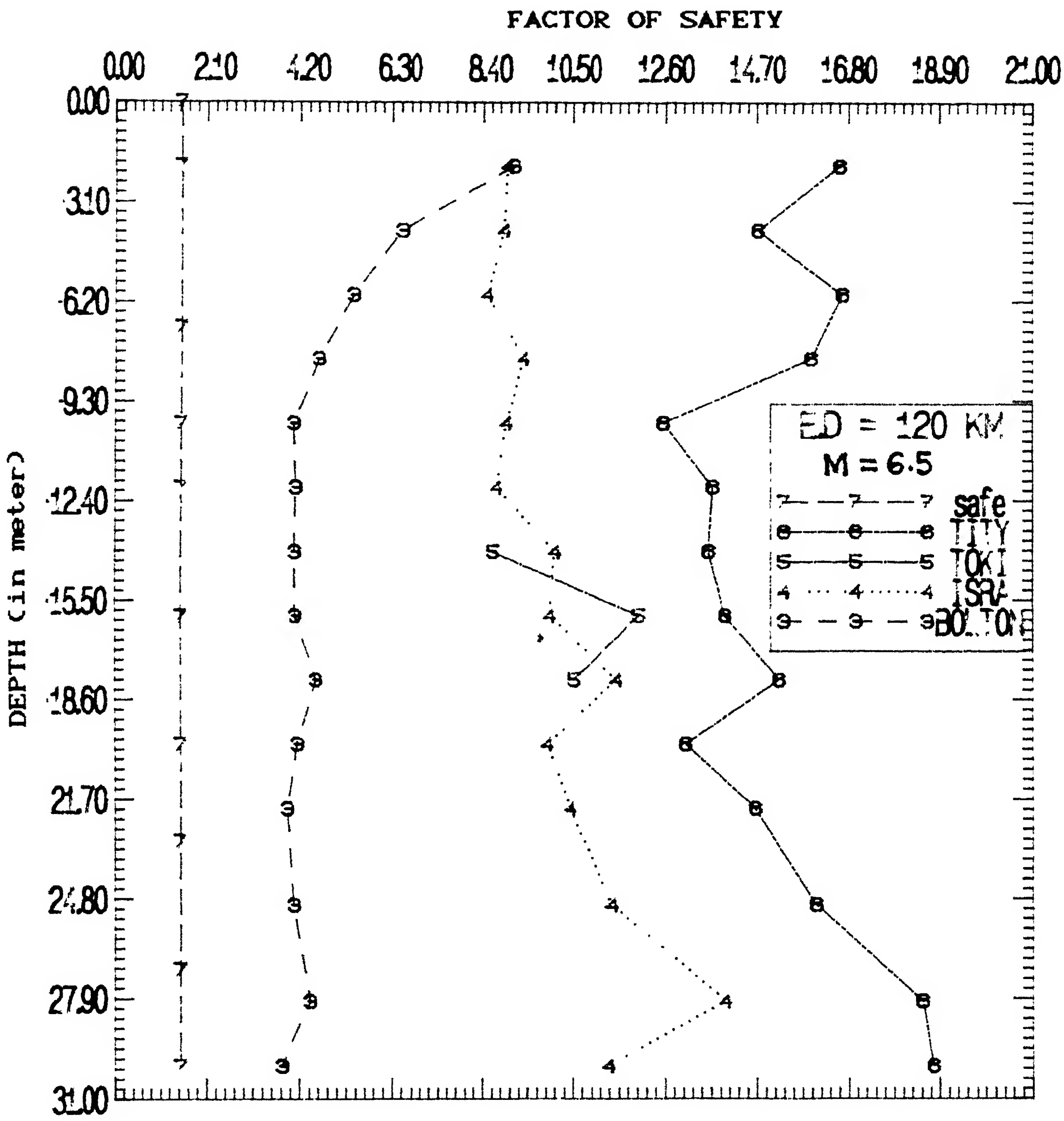

FIG-4.3.14 **RELATIONSHIP BETWEEN FACTOR OF SAFETY AND DEPTH (KANPUR)**

limited depth. Chang method predict that soil would not liquefy at all.

All the methods predicts sufficiently large value of factor of safety against liquefaction. The scatter in the predicted values are quite large. It is the safest against liquefaction of all the considered sites.

A salient findings of the study is presented in table.

SUMMARY TABLE

SITE	M	E.D Km	A_{max}/g	METHODS	Liquefy or Not	ZONE (metre)
GORAKHPUR	8	180	0.045	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	×	
				CHANG	YES	2 - 20
	7	150	0.07	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	×	
				CHANG	YES	0 - 10
DEORIA	8	180	0.045	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	×	
				CHANG	YES	0 - 10
	7	150	0.07	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	×	
				CHANG	YES	0 - 10
RAIBARELY	6	15	0.15	BOLTON	YES	10 - 14
				ISRA	NO	
				TITY	NO	
				TOKI	×	
				SHITE	×	
				CHANG	NO	
KANPUR	6.5	120	0.04	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	×	
				CHANG	NO	
SHAHJAHNPUR	8	125	0.09	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	YES	0 - 12
				CHANG	YES	4.5 -12, 19 - 23

SHAHJAHNPUR	6	15	0.15	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	YES	0 - 15
				CHANG	NO	
ANOLA	8	125	0.09	BOLTON	NO	
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	YES	0 - 13
				CHANG	NO	
	6	15	0.15	BOLTON	YES	12
				ISRA	NO	
				TITY	NO	
				TOKI	NO	
				SHITE	YES	0 - 14.5
				CHANG	YES	0 - 12

TABLE-4.1

Based on the results and discussions the following generalized conclusions are drawn:

(a) Chang's predictive model(1989) for soil liquefaction is in general the most conservative. Shibata and Teparaksa method(1988) is also very conservative but this conclusion can not be generalised as the method could only be applied to only two sites. Bolton and Idriss method(1971) also predicts factor of safety values on the conservative side.

Predictive models proposed by Tokimatsu and Yoshmi(1983), Tatsuoka et al.(1980) and Ishihara1979) predict large factor of safety values.

As such, in the absence of definite correlations for Indian Soils it is prudent to use the three methods proposed by Chang(1989), Shibata and Teparaksa(1988), Bolton and Idriss(1971) for predicting the factor of safety against soil liquefaction and then choose the least one to be on the safer side.

(b)The sites are sequentially placed in the decreasing order of the risk of soil liquefaction. They are Anola,Shahjahnpur, Deoria, Gorakhpur, Raibarely, Kanpur.

(c) The difference in the assessment of liquefaction potential by different models is probably due to refinement in the predictive models from time to time giving rise to different input parameters based on which the risk evaluation has been made. Even though some qualitative assessment can be made about the conservativeness of a method, it is very difficult to find which one is the least conservative.

4.5 Scope of future study:

The following topics are suggested for further research,

1.Since all the available correlation for evaluating the cyclic shear strength, necessary for predicting liquefaction potential may not be valid for local deposits, it is suggested that cyclic strength be evaluated in the laboratory and semi-empirical correlation be developed for such deposits.

2.SPT and CPT be conducted at different locations in deposits likely to liquefy and mapping of liquefaction potential of the various area of country be made.

3.Immediately after the occurrence of earthquakes, in situ tests like SPT and CPT be conducted in the affected areas for better understanding and verification of the developed predictive models for the local deposits.

4.Analytical models for the pore pressure build up caused by random loading during earthquakes be made and validated for proper estimation of effective stress.

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